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Analysis of river scour at bridges in Southwestern Ontario.

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ANALYSIS OF RIVER SCOUR
AT BRIDGES
IN SOUTHWESTERN ONTARIO

A THESIS

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by

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ABSTRACT

In this thesis, scour of river bed material at several bridge sites in Southwestern Ontario is investigated. The data was taken from sites where scour had occured.

Dimensional analysis is applied to establish dimensionless parameters. From these dimensionless ratios, several trends of the scour phenomena were found. A comparison of critical velocity to actual velocity as well as critical boundary shear stress to actual boundary shear stress was made at the stations being investigated.

Comparisons were made of the actual depth of scour to the depth given by Laursen's formulae as well as to the depth given by applying Blench's Regime Theory.

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CHAPTER 1

INTRODUCTION

Bridges are often constructed in positions where scour and erosion are very serious problems. Bridges at one time were built where the channel of the river was straight and the banks stable; now highways are being built with minimum mileage as the controlling factor. Thus the bridge is often placed in poor soil, at an angle or on a river bend. Where the problem of scour arises, it is advantageous to know how much scour will occur. Scour is especially important at the foundations. As can be expected, the superstructure of a bridge can be seriously damaged by a relatively small amount of pier settlement brought on by erosion. Since the cost of pier foundations is generally high and increases rapidly with depth, the problem is also one of economy. Thus the depth of possible scour should be known for economy as well as protection for the foundations.

The complex features of natural processes involved in scour make it difficult to study by direct deductive reasoning. The depth of scour depends on many conditions, such as the slope of the river, the particle size of the bed material, the velocity and depth of the water and other factors that may be related to the tractive force. It would be very difficult to start with a basic physical law and from this arrive at an equation that is not empirical. Rather it is necessary to start with a mass of observed facts, and from these establish a pattern to govern the data being studied. Such a study would involve rivers of a certain size or geological age in a specific area.

In Southwestern Ontario, the problem of scour is very important because the maximum flood flow is often many times greater than the mean flow. It is common to find the flood flow 20 or 30 times greater than the mean flow. Design for bridges over rivers of high flood depends on an accurate estimation of the depth of scour to be expected for the maximum flood flow of the river.

In the field of scour prediction, there are several formulae existing. It is believed that the magnitude of the shear stress is a very good indication as to whether scour is occuring.

This thesis is based on a study of scour at bridge sites, sponsored by the Department of Highways of Ontario and designated as Project A-1. The study was conducted under Dr. H. Herbich of The University of Windsor.

CHAPTER 11

SCOUR AT BRIDGE PIERS AND ABUTMENTS

Obstruction by the bridge pier will modify the stream velocity pattern and cause scour at the bridge if the conditions before the bridge are close to critical shear stress. Several general characteristics are common to all the scour patterns that occur around piers. First of all, the upstream portion of the hole has the approximate form of an inverted cone, sometimes distorted from the circular. Second, the side slope of this cone is close to the angle of repose of the soil in which the scour is occuring. Third, the zone of greatest depth of scour is generally displaced from the face of the pier.

Where the pier is of two shafts, a separate scour hole will be formed at each shaft (11). At small angles of approach (0 to 10 degrees), the downstream shaft is shielded by the upstream shaft and accordingly it has a shallower scour hole. As the angle or approach increases, not only is the protection from the upstream shaft lost, but the downstream shaft becomes subject to the currents of higher velocity produced by the shaft upstream and results in a shift of region of deepest scour to the downstream hole. However, at large enough angles, this interference effect tends to disappear.

The effect of different types of shafts in the pier construction is also very important. The most pronounced difference between the effects of round and rectangular shafts is observable at small angles. Because of its sharp edges, a rectangular shaft will cause greater disturbances than a circular shaft. The depth of scour caused by a rectangular shaft will be about 15 percent greater than that of a circular shaft.

The scour hole that occurs at the abutment of a bridge is similar to that for piers, the only difference being scour occurs along the flow path.

APPLICATION OF THE REGIME THEORY

Most rivers at some stage of flow, move material and thus the water is not pure. Here the principles of conventional hydraulics are not sufficient in describing the situation. The rivers are influenced by the laws of hydraulics and also by laws of sediment transport and of erosive resistance of banks. The river adjusts to increasing or decreasing flow by changing its width, depth, slope and meander pattern. Interference, such as a bridge, will cause the river to adjust to reimpose either the original regime or another that is consistent with the obstruction.

Material in the water of the river falls in two classifications. Some material will be carried without resting on the bed, though it may strike the bed. This material is called the suspended load. Material may also move in a manner that can be called rolling or hopping. This material is called the bed load and is in contact with the bed. The bed load moves along the stream bottom in the form of moving dunes. The particles move by trickling up the dunes and falling to the next dune where the impact may cause or help other grains to start moving; this type of movement is called saltation. At high velocities the dunes vanish and transport is in a sheet of saltating particles.

REGIME AND DEGREES OF FREEDOM (4)

Consider a discharge of fluid passed down a laboratory flume. Kinematically flow at any depth will have a corresponding velocity. The flow stays at one depth to suit the law of hydraulic resistance, and this law is expressed in any rigid boundary hydraulics formulae. Here the channel has one degree of freedom, or self-adjustment of its free fluid surface.

If a bed load of sand is injected along with the fluid, part of the sediment will deposit on the bed and build up a slope steeper than that of the flume. Deposition will stop when the bed of moving sediment has acquired a slope that permits bed load at all sections to be the same. Here the depth adjusts to suit the resistance law of the moving bed combined with rigid sides and the slope adjusts to suit the law of bed load transport. This channel has two degrees of freedom.

If the flume is replaced by a canal cut in cohesive erodible soil and flow conditions are severe, the sides will erode until a balance is reached. If a suspended load is introduced to a channel with large width, the suspended material will adhere to the sides and cause them to grow inward to reduce the width. This channel has a third degree of freedom and the width is adjusted to suit the law concerning cohesive resistance to erosive attack.

If maintenance of the canal in the erodible soil is neglected for a relatively long period of time, small irregularities of the sides will develop and show that the artificial straight form is really unstable. A fourth degree of freedom exists that is associated with erosive attack of curved flow.

DISCUSSION OF REGIME

In the cases above, a steady condition will be reached at different times. In the first case it will occur several seconds after the flow is turned on. In the second, a few days will be needed before the sediment load entering the head of the flume equals that coming out at the exit. In the third, a year or two would be needed for a channel length of about 50 miles. In the fourth, of a neglected canal turning into a "river", the whole meander sequence might take decades to reach steadiness. A natural river with its enormously varying discharge from day to day would take even longer; its "steady" condition is of steadiness about a mean state, just as the steady velocity in turbulent flow is really a steadiness of an average about which erratic fluctuations occur indefinitely. If imposed conditions are fixed, or oscillate about equilibrium, then steadiness or equilibrium must result. River and canal engineers have used the word "regime" to replace "equilibrium" which is associated with laboratory physics. "Regime" suggests considerable freedom of individual behavior within a framework of laws and has no short-period connotation. The concept of the degrees of freedom is necessary for the engineer proposing to interfere with a river in "regime".

Blench (4) set forth three basic regime equations to represent ideal behavior in channel flow. All channels will adjust to the same value of $\frac{V_a^2}{d}$ if they have the same water-sediment complex. This is known as the bed factor and as an equation is expressed as

$$F_b = \frac{V_a^2}{d}$$

The side-factor equation is given as

$$F_s = \frac{V_a^3}{b}$$

where "b" is the width at about half depth. If "F_s" exceeds a certain upper limit for a given bank material then the channel will widen until "F_s" drops to that limit; if "F_s" falls short of a certain lower limit then suspended load will deposit on the banks and reduce the width until the lower limit is reached.

The third basic regime equation is the slope equation and is given as

$$\frac{V_a^2}{g\,d\,S} = 3.63\left(1 + \frac{C}{233}\right)\left(\frac{V_a b}{\nu}\right)^{.25}$$

in which S is the channel slope, expressed as a fraction, C is the bed-load charge in parts per hundred thousand by weight and ν is the kinematic viscosity of the water-sediment complex.

The engineer knows the discharge to expect and can estimate the bed factor and side factor. The regime equations thus can be rewritten in useful form.

$$\text{Width} \quad b = \sqrt{\frac{F_b Q}{F_s}}$$

$$\text{Depth} \qquad d = \left(\frac{F_s Q}{F_b^{2}}\right)^{\frac{1}{3}}$$

$$\text{Slope} \qquad S = \frac{F_b^{\frac{5}{6}} \; F_s^{\frac{1}{12}}}{\left(1 + \frac{c}{233}\right) K \; Q^{\frac{1}{6}}}$$

The value of K is about 1,800 in cold areas and 2,000 in hot areas.

Q = Discharge in c.f.s.

c = parts of suspended material per litre of water

Since the bed factor varies with charge, it cannot be readily estimated. Thus a zero bed factor is introduced; this is the bed factor at a very small stage and F_{bo} is a measure of the bed material only. The small stage occurs when the rating discharge is zero.

A close estimation for the zero bed factor can be given by

$$F_{bo} = 1.9 \left(D_{50}^{\frac{1}{2}}\right) \qquad \text{(for sands)}$$

$$F_{bo} \propto D_{50}^{\frac{1}{3}} \qquad \text{(for gravels)}$$

Where D_{50} is the median diameter in millimeters.

Regarding the zero bed factor F_{bo}, a graph has been published recently by Blench and Qureshi (14) showing zero bed factor values for bed materials ranging from fine sand to boulders. This is seen in Figure 1.

Values of F_{bo} are found to vary from .5 for fine sands to 6. for small boulders. The values for the rivers studied in Project A-1 fall within these limits.

For the Blench Regime Theory for flood flow between and around bridge piers, Sanden (17) suggested the multiple 1.8. The formula is written as

$$d = \frac{1.8\ (q)^{\frac{2}{3}}}{(F_{bo})^{\frac{1}{3}}}$$

with $F_{bo} = 1.9\ (D_{50})^{\frac{1}{2}}$

where d = the depth of scour measured in feet from the water surface

" q = the unit discharge (defined at half depth)

" F_{bo} = the zero bed factor

" D_{50} = the median particle diameter in millimeters in terms of weight of an average sand bed sample.

The unit discharge was taken as the total discharge divided by the width of the river at its surface.

In the rivers being studied in Project A-1, the actual depth of scour was known. Application of the Blench formula gave depths that were greater than the actual for all cases. To find the formula that would satisfy the Southwestern Ontario area, it was felt that the numerical coefficient should be changed as it did not appear to be constant. In each case the actual coefficient was calculated because the actual average depth was known; this value was then plotted against the corresponding actual depth and plotted on graph paper. In Figure 2, it can be seen that the value of the calculated coefficient decreases as the average depth increases.

Using the Correlation Theory, the regression line of the calculated coefficient C_c on the Depth d was found to be

$$C_c = 1.42 - .0359\ d$$

Thus the Regime Theory for Southwestern Ontario becomes

$$d = \frac{(1.42 - .0359\ d) \times (q)^{\frac{2}{3}}}{1.24 \quad (D_{50})^{\frac{1}{6}}}$$

By solving for d we get

$$d = \left(\frac{1.42}{\frac{1.24 \quad (D_{50})^{\frac{1}{6}}}{q^{\frac{2}{3}}} + .0359} \right)$$

It should be noted that the range of the discharge is only up to 20,000 cfs, which is far below the range for discharge of the rivers studied by Sanden. Sanden mentioned that there was difficulty in measuring the depth of scour at high flow and velocity values. The depth and velocity measurements used in Project A-1 are considered fairly accurate, but the soil particle size may be in error because of the lapse in time between the measurement of velocity-depth data and the sampling.

The particle sizes are listed in Table 1.

The Correlation Coefficient for the function derived from the values on Figure 2 is .713.

CHAPTER III

SCOUR RELATIONS

Unit Flow and Scour Depth Relations

When the flow is high enough to induce scour, the maximum depth of scour will be reached in time.

The relationship for the depth of scour holes below hydraulic works is expressed by Ahmad (1) as

$$\frac{(d_s)^{\frac{3}{2}}}{q} = C$$

Where d_s is the maximum depth of scour, q is the unit discharge and C is a constant that differs for each situation.

This formula applied to the Thames River at Thamesville and the Nottawasaga River at Baxter gives a method of estimating scour at these bridge sites. Using the data for the Thames River at Thamesville on the date of March 28, 1963, the constant is

$$C = \frac{(d_s)^{\frac{3}{2}}}{q} = \frac{(3.00)^{\frac{3}{2}}}{90} = .0653$$

For the Nottawasaga River at Baxter on the date of April 4, 1960

$$C = \frac{(d_s)^{\frac{3}{2}}}{q} = \frac{(4.63)^{\frac{3}{2}}}{46.1} = .216$$

Figure 3 illustrates these functions.

As can be seen by the plot of these two curves, low values of q will have corresponding values for d_s. This is not true because deposition or no action will occur in this low range. No attempt was made in Project A-1 to find the range for which this constant C can be used.

EFFECT OF FLOW CONTRACTION

It can be expected that, in general, contraction of a cross section by piers will result in an increased depth of scour. Certainly for a long contraction, the bed will be scoured to a lower elevation than in the uncontracted flow. A solution for scour in a long contraction was presented by Straub (12). His solution can be written as

$$\frac{d_s}{y} = \frac{1}{(1 - B)^{9/14} - 1}$$

in which (1 - B) = the ratio of the contracted to the uncontracted width

where d_s = the equilibrium depth of scour measured from the normal bed (feet)

y = depth of flow (feet)

Straub noted that the contraction of the flow section does not have any effect on the scour around piers until the depth of scour at the pier approaches the depth of scour which would occur in a long contraction. As the spacing between piers becomes smaller, the depth of scour can be approximated by Straub's solution.

Contraction has to be in the order of 10% by the piers before the depth of scour will be affected by contraction. The amount of contraction due to bridge piers varies from bridge to bridge.

With consideration to constriction, the ratio of the depth of scour to the depth of water by Laursen (12) is given as

$$\frac{d_s}{d} = \left(\frac{B_1}{B_2}\right)^n - 1$$

$$n = .59 \quad \text{for } \frac{v_s}{v_f} < .50$$

$$n = .64 \quad \text{for } \frac{v_s}{v_f} = 1.0$$

$$n = .69 \quad \text{for } \frac{v_s}{v_f} < 2.0$$

where $\frac{B_1}{B_2}$ = the constriction ratio

$\frac{v_s}{v_f}$ = the shear velocity to full velocity ratio.

The constriction ratio was taken from aerial photographs of the river stations being studied. The actual depth of scour to the depth of water ratio is listed with that calculated by the Laursen formula in Table 2. The measurements from the aerial photos are estimations as the photos were not necessarily taken at flood periods.

CHAPTER IV

INVERTED FILTERS

The foundation of a bridge pier must rest on fairly firm ground so it will not be undermined by running water. Protection against erosion can be accomplished by layers of uniformly graded gravel or stone being placed where erosion may occur. Also, transitory and fixed pressure from the flow can induce sub-hydrostatic pressures in the material and also cause erosion. Patterns of fixed pressure differences remain constant as long as the bottom maintains a given configuration; these will change if the bottom shifts. The importance of these pressure differences lies in the fact that the flow induced through the porous bed material is upward at low pressure areas; this means that erosion tends to be rapid at these areas. Any attempt to protect an erodible bed at a location of fixed subhydrostatic pressure must be made with material that is pervious or that is fully capable of remaining in place at these pressures. Thus erosion protection must be designed to act as an inverted filter.

A filter is used so that the creep of the grains is prevented when water percolates through it. The final layer of blocks or boulders to be placed on the filter bed must be large enough to withstand the force of flowing water. To deposit the different layers of a true filter is very expensive and complicated, when the work must be carried out in running water. Attempts were made to prevent erosion by pouring gravel into the water and covering it with blocks of stone; often this proved useless since the grain size distribution of the gravel and the proper dimension of the armour layer are very important for obtaining stability of the filter.

The required stone diameter necessary to insure stability of an armour layer in running water is given by Andersson (3). The first equation is based on shear stress and the second on shear velocity. They are as follows:

$$k = \frac{S_f}{S_s - S_f} \cdot \frac{Z \sin\alpha}{C} \cdot \frac{1}{\tan\phi \cos\alpha - \sin\alpha}$$

$$k = \frac{S_f}{S_s - S_f} \cdot \frac{V_m^2}{K\left(\log \frac{14.83 Z}{k}\right)^2} \cdot \frac{1}{\tan\phi \cos\alpha - \sin\alpha}$$

where k = diameter of the stone in centimeters

S_f = unit weight of water

S_s = unit weight of stone

z = depth of water (meters)

α = angle of slope in direction of flow

ϕ = angle of natural slope of the stone

V_m = mean velocity of water in meters per second

C and K = values to be found on Figure 4a and Figure 4b.

N = the degree of erosion or percent of the surface layer that has been moved.

Application of the velocity formula by Hedar (6) establishes K $k = 7.5$ and $\tan\phi = 1.11$. The Andersson (3) formula based on velocity can be rewritten as

$$k = \frac{S_f}{S_s - S_f} \cdot \frac{V_m^2}{7.5 \left(\frac{\log 14.38 z}{k}\right)^2 (1.11 \cos\alpha - \sin\alpha)}$$

A filter can be made by substituting a single mixed filter containing various grain sizes and stones of a certain thickness rather than an expensive filter of individual layers of uniformily graded material. After the material is placed, the fine material on the surface of the mixed filter will be washed out by the action of the water, which will reduce the thickness of the filter. The further the water penetrates into the interstices between the blocks, the more its velocity is reduced. The loss of fine material continues until the mixed filter consists, at a given depth, of particles large enough to resist the action of water at that depth. Cross-sections of the two filters are seen in Figure 5.

Criteria for another method of filter construction have been given by K. Terzaghi and tested by the U. S. Waterways. The specifications for the filter called Terzaghi-Vicksburg are as follows:

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{Base}} < 5$$

$$4 < \frac{D_{15}\ \text{Filter}}{D_{15}\ \text{Base}} < 20$$

$$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{Base}} < 25$$

i.e. Fifteen percent, by weight of the filter material is finer than the size indicated by D_{15} Filter.

Tests by U. S. Waterways show that a 2" layer of this filter was more effective than an 8" layer of uniform material of the size of the largest particle in the 2" filter.

Generally, stable rivers have bed material that has a wide range of sizes. The presence of a uniform bed material size indicates deposition which has followed a previous period of scour. Thus it can be seen that the mixed layer is more resistant to erosion, due at least to subhydrostatic pressure.

The specifications for the Terzaghi-Vicksburg mix appear to be complicated and expensive; actually it is much cheaper than uniform material since the desired grading probably can be obtained by very little sieving or blending.

CHAPTER V

COLLECTION OF DATA

To study the phenomena of scour at bridges, many variables have to be taken into consideration. As Project A-1 was sponsored by the Department of Highways of Ontario, the study was conducted on Ontario rivers where severe scour occurs almost yearly. The choice of rivers was made with the assistance of the staff of the Water Resources Branch of the Department of National Resources and Northern Affairs located in Guelph. The information of depth measurements, velocity measurements, daily flows, and rating curves was taken from the files of the Water Resources Branch in Guelph and most of the calculations are based on these measurements. The map shown in Figure 6 shows the stations considered in this study. Table 3 lists the name of each station and the number assigned for identification on the map. Additional information of soil size and river slope was obtained on a survey conducted by the staff of the Civil Engineering Department of the University of Windsor that was involved in Project A-1. Several soil samples were taken at each location in order to obtain representative average size distribution. The samples are of the material that is subject to scour. In several cases, a layer known as the "armour coat" had to be removed before sampling could be done. The "armour coat" is a layer of larger boulders remaining at the stream bed after the finer material has been scoured. The samples weighed approximately 10 lbs. each and were placed directly into canvass bags. Most samples tended to be granular rather than clay. The depth of sampling varied from one to two feet.

PARTICLE SIZE

To determine the particle size, each sample was tested. The samples were oven dried at 105° Farenheit for approximately 24 hours before being tested. Varying sets of sieves ranging from #200 to #3 were used to determine the particle size distribution. Where the percent finer than #200 was greater than 12, a hydrometer analysis was used to determine the size distribution below #200. The results were plotted on Grain Size Distribution Graphs. Values for the 10, 25, 35 and 50 percent retained were read from the graphs and average values for each of the stations were calculated.

The average particle sizes are listed in Table 4 and samples of the Grain Size Distribution Graphs are seen in Figure 7 and Figure 8.

River bed material that is uniform in size indicates cyclic scour and deposition. The samples taken for the study, all indicated the presence of at least 30% being of a uniform type of material.

The Specific Gravity for the samples was also determined in the University Soils Laboratory. The results are listed in Table 5.

BOUNDARY ROUGHNESS COEFFICIENT

From the particle size, the boundary roughness coefficient n was calculated. The most universally used discharge formula for open channels in North America is that of Manning:

$$Q = \frac{1.49}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$$

where n is the boundary roughness coefficient, Q is the discharge in c.f.s., A is the cross-section area in square feet, R is the hydraulic radius in feet, and S is the slope of the river bed. These factors and others are considered characteristic of each location and may influence scour.

The value of n also will be applied to the method outlined by Chow(5) for computing shear stress.

Several formulas presently are being employed for determining the value of n. Strickler (19) arrived at a formula based on empirical studies made in Switzerland in 1923 which related Manning's n and the median size of bed material D_{50} in inches. His formula is

$$n = \frac{(D_{50})^{\frac{1}{6}}}{44.4}$$

A similar formula was proposed by Keulegan (19). The difference is the denominator constant

$$n = \frac{(D_{50})^{\frac{1}{6}}}{46.9}$$

where (D_{50}) is also in inches.

A relationship between Manning's roughness coefficient and the size of the bed material was also given by Irmay (19). Instead of using the median diameter, Irmay used the maximum. Where the bed is made up of a mixture of sizes, the maximum size is taken as the particle size at the 10% retained value. His relationship is

$$n = \frac{(D_{10})^{\frac{1}{6}}}{49.0}$$

where (D_{10}) is in inches.

Lane and Carlson (10) arrived at one formula based on the knowledge of the natural bank material. They took into consideration the particle at the 25% retained by weight value. This relationship is

$$n = \frac{(D_{25})^{\frac{1}{6}}}{39}$$

where (D_{25}) is the size in inches of which 25% is larger by weight. A plot showing the relationship between relative roughness $\frac{D_{35}}{R_H}^{\frac{1}{6}}$ and Manning's n also gave Lane and Carlson another formula:

$$n = \frac{(D_{35})^{\frac{1}{6}}}{R_H} \quad \frac{1}{26}$$

where (D_{35}) is the size in feet of which 35% is larger by weight and n is the hydraulic radius in feet.

Straub (18) used the median particle size for the calculation of n

$$n = .0432\left(D_{50}\right)^{\frac{1}{6}}$$

where (D_{50}) is the 50% size of the bed material in feet.

These formulae were applied to each station under consideration. The individual values of n varied very little at each station and there was no reason to justify the use of any single equation. These values are in Project A-1. For a characteristic value of n, the average was taken. The boundary roughness coefficients are listed in Table 5.

On the field trip, the slope at each station was taken. This was done with a leveling instrument for a distance of about 300 feet, both above and below each station. The geometric average was found by taking the square root of the product of the slopes above and below. As the measurements were taken at a time of no flood flows, it is felt that the measurements are fairly representative.

Slope of a river surface generally decreases as the water rises. However, the opposite occurs during flood flow. The slope increases as the river rises. This is because in a short time the water swells rapidly and is referred to as the rising stage.

This was checked with information taken at two stations, Thames River at Thamesville and Nottawasaga River at Baxter. For each station, the slope versus flow and slope versus stage were plotted on log-log paper. Extrapolation gave the same results in slope for flood flow measurements at both rivers. Several authors give formulae for estimating the slope. It is felt that slope change is characteristic for each cross section point, similar to that of a rating curve. This is because coincidental results are found by extrapolation on the above mentioned plots on log-log paper. The information and results are listed on Table 6.

It is known that stream-discharge rating curves can be subject to shifting and change, as the river bed and the side slopes are subjected to scour and deposit.

Ice in any form at a control results in a condition of shifting control (13). Anchor ice displaces the control by an amount equal to the ice thickness. Sheet ice provides a new friction surface and thus reduces the channel capacity from that for a corresponding open-river stage. Also stage readings may change due to vegetation growth or stage-discharge looping. Looping is due mostly to a change of slope from the rising to the falling limb of the flood. Such a loop was found for the Thames River at Thamesville in the flood of 1963.

The fact that the slope has been extrapolated as the same for the Stage versus Slope and Flow versus Slope graphs is a confirmation of the steadiness of the rating curve. However, limited information prohibits any conclusions to be drawn from this observation.

It may be mentioned in relation to the flood flow that Karuks (8) sets forth a method of flood prediction. It is based on several factors characteristic to each watershed but does not cover tropical storms.

CHAPTER VI

CRITICAL AND ACTUAL SHEAR STRESS

During the development of scour prediction, authors have related the depth of scour to many different factors. Some authors felt that scour depended on the water depth only while others used the particle size for estimating scour. Recent literature indicates that scour is caused by a force that is built up on the stream bed. Water flowing in a channel develops this force on the bed in the direction of flow. This force is called the tractive force. The boundary shear stress is the unit tractive force or the force per unit area of the wetted channel bed. The critical shear stress is the maximum a given channel can withstand; by definition any value above this will cause erosion. The critical shear stress value can be found for a sample by laboratory experiments. However, coarse non-cohesive material in a stream bed can stand higher values than the critical shear stress found in the laboratory. This is probably due to slight amounts of colloidal and organic matter found in the water and soil. The actual shear stress can be calculated from vertical velocity profiles taken from current-meter measurements. When scour occurs, the finer particles are suspended first. When the river flood recedes, differential settlement occurs with the heavier particles settling out first. Most settlement occurs just after the flood period, and it tapers off into the summer months. Such settlement has made sampling difficult and care had to be taken to extract the type of material that was in contact with the flowing river at the time of flood.

To establish the actual boundary shear stress it is necessary to obtain discharge and vertical velocity distribution data close to the flood peak. In this study, the measurements are representative of flood periods.

A comparison of actual and critical shear stress will later be made to determine whether the actual is higher than the critical in the cases of scour. This comparison can not be done for the minimum flow records as differential velocity measurements for these flows do not exist.

With the formulae described in this chapter, calculations for the shear stress will be made.

SHEAR STRESS RELATED TO SCOUR

The concept of tractive force was introduced by DuBois in 1879. Shear stress on the bed of an alluvial channel of infinite width and uniform flow was given as

$$\tau_0 = \gamma d \quad S$$

Following DuBois, Chezy developed an equation for shear stress for a channel of finite width and irregular shape. This equation for unit shear stress is

$$\tau_c = \gamma R \quad S$$

The shear stress is not uniformily distributed along the wetted perimeter of most rivers or channels. An estimation for design is often used by engineers; for channel shapes similar to that of Figure 11, the maximum shear stress on the bottom is equal to .97 d S and on the sides is equal to .76 d S.

According to Chow (5) the soil particle size has to be taken into consideration for determining the critical shear stress. On a soil particle resting on a sloping side of a channel section, (Figure 11) in which water is flowing, two forces are acting; the tractive force ($a\tau_s$) and the gravity force component ($W_s \sin\phi$) which tends to cause the particle to roll down the side slope.

a = the effective area of the particle

τ_s = the shear stress on the side of the channel

W_s = the submerged weight of the particle

ϕ = the angle of the side slope.

The resultant of these two forces is equal to

$$\sqrt{W_s^2 \quad \sin^2\phi \quad + \quad a^2\tau_s^2}$$

When this resultant force is large enough, the particle will move. When motion is impending the resistance to motion of the particle is equal to the force tending to cause the motion. The resistance to motion of the particle is equal to the normal force $W_s \cos\phi \tan\theta$

where θ = the angle of repose

$\tan\theta$ = the coefficient of resistance

Equating the two forces and solving for the shear stress

$$\tau_s = \frac{W_s}{a} \cos\phi \tan\theta \sqrt{1 - \frac{\tan^2\phi}{\tan\theta}}$$

On a level surface, the value for is zero and the shear stress at impending motion on a level surface is

$$\tau_L = \frac{W_s}{d} \tan\theta$$

The ratio of τ_s to τ_L is called the tractive force ratio K and is often used in shear calculations.

$$K = \frac{\tau_s}{\tau_L} = \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}}$$

This tractive force ratio is a function of the inclination of the side and the angle of repose of the bed material. For cohesive and fine non-cohesive materials, the cohesive force becomes so great in proportion to the gravity force component that the gravity force can be neglected.

Therefore, the angle of repose need only be considered for coarse non-cohesive materials. The U.S. Bureau of Reclamation has found that the angle of repose generally increases with particle size.

Curves of the U.S. Bureau of Reclamation (18) and of the U.S.S.R. (18) relating the Permissible unit tractive force to the bed material are shown on Figure 12.

The approach used by Stebbings (18) is similar to that of Chow (5). Stebbings included channel slope in his study. The component of weight normal to the bed surface will be $m\ g\ \cos\phi_s \cos\theta$ and the limiting friction resistance is $m\ g\ \cos\phi \cos\theta \tan\phi_o$, where $\tan\phi_o$ is the angle of internal friction of the material. Equating resultant motive force to frictional resistance:

$$\sqrt{m^2 g^2 \sin^2\phi_s + a^2 \tau_c^2} = m\ g\ \cos\phi_s \cos\theta \tan\phi_o$$

Solving this, Stebbings arrives at the formula

$$\tau_c = \frac{m\ g}{a} \tan\phi_o$$

where τ_c = critical shear stress

m = Mass of individual sand particle

g = Gravitational acceleration

a = Area of sand particle over which shear stress acts.

ϕ_o = angle of repose of sand.

This shows that the maximum shear stress depends on the sand characteristics and is independent of the channel dimensions and slope.

Extensive studies carried out by the Bureau of Reclamation have stressed the use of shear in the design of canals for stability against scour. Lane and Carlson (10) conducted studies for the Bureau in the San Luis Valley. It was found that the canals in the San Luis Valley had material extending from fine sand on up to gravel and that the finer material had been washed from the top layer of the bed leaving an armour coat. Lane and Carlson had hoped that all the material below a certain size would be found to have been removed from the bed. However, this was not the case - no satisfactory analysis of the data based on specific size left in the bed was found.

Thus it was decided by Lane and Carlson to relate shear stress produced by maximum flow to the particle size of the material through which the canal was originally constructed. This relationship is advantageous because samples of material through which a canal is proposed can be easily obtained.

An arbitrary parameter was adopted to describe the natural material; this was taken to be the sieve size of the grading of which 25 percent of the weight of the material is larger. By plotting the 25 percent larger size of the natural material against the tractive force, a relation was found which can be used for design and for shear stress calculations. The limiting shear stress in pounds per square foot is equal to four-tenths of the size in inches of the sieve opening on which 25 percent of the weight of the natural bank material will be retained. As an equation this is

$$\tau_c = .4 \ (D_{25}) \text{ in}$$

Consideration was given to the particle size and the difference in specific weight of the bed material and the water. A relation was given by White in 1936 (20) and is expressed as

$$\tau_c = .18 \ (\gamma_s - \gamma_w) \cdot D_{50} \tan \phi$$

where γ_s = the specific weight of the soil

γ_w = the specific weight of the fluid

D_{50} = 50% passing size (by wt.)

ϕ = angle of repose,

τ_c = critical shear stress .

Shields felt the angle of repose can be neglected. The equation of Shields is written as

$$\tau_c = .06 \ (\gamma_s - \gamma_w) \cdot D_{50} \ .$$

The particles were assumed to be spherical and the resultant force was assumed to pass through the centre of the grain.

Experiments by Tison (20) similar to those of White and

Shields conclude that the quotient in the formula can vary, depending on the Reynolds Number, N_R. In the case of $N_R < 3.5$, the value of τ_c determined by the formulae of White and Shields show that the magnitudes of the shear stress should be divided by a factor ranging between 2.46 and 6. This gives the quotient a range between $\frac{.18}{N_R}$ and .03, but not exceeding .09. Tison had not verified this and believed that the quotient depends not only on the N_R but also on the depth of water.

In the case where $N_R > 3.5$ the factor varies between 4 and 7 in the region of $3.5 < N_R < 70$. (the highest value corresponding to low N_R). For shear stress Tison gives a range shown in this formula

$$\tau_c = .03 \longrightarrow .07 \ (\gamma_s - \gamma_w) \ D_{50}.$$

For calculating the actual shear stress from the vertical velocity profile, the Prandtl-Karman law (2) of logarithmic velocity distribution was used. The velocity distribution in a uniform channel flow is stable when the turbulent boundary layer is fully developed. The shear stress at any point in a turbulent flow moving over a solid surface is given as

$$\tau_a = \rho \, l \left(\frac{(d\ v)}{(d\ y)}\right)^2.$$

where ρ = mass density

l = mixing length

$\frac{d\ v}{d\ y}$ = velocity gradient at a normal distance y from the solid surface.

Prandtl assumed the mixing length to be proportional to y

and in the turbulent area the distribution can be shown to be approximately logarithmic. Thus the shear stress is

$$\tau_a = \rho \left(\frac{v_2 - v_1}{5.75 \log \frac{y_2}{y_1}} \right)^2 .$$

The subscripts 1 and 2 indicate two different points of depth at which the velocity was measured. For stream flow measuring the general practice is to divide the cross section into 15 or 20 increments and take veloctiy soundings at the .2 and .8 depth at each increment.

In calculating the values of the critical shear stress for the above equations it was found that the values deviated little from each other for each station. Because of this the average value of the critical shear stress, rather than that of any specific author, was used. The actual boundary shear stress was calculated with the Prandtl equation. The information of the vertical velocity profile was taken from field stream-gaging data which were available at the Water Resources Branch in Guelph. The data taken from Guelph for the sample stations are listed in the first five columns of Table 9 and Table 10. The area and depth of scour can be seen in the sample cross section diagrams in Figure 15 and Figure 16. The cross sections compare the flooded river bed to the bed outline of the dry period just before the flood. In all cases the river scoured the river bed. A tabulation of boundary shear stress is made in Table 7, in order that a comparison of the actual and critical condition can be compared. The cross section areas have scoured in the flood period and the corresponding shear stress at flood time is greater than the critical shear stress for all stations.

The DuBois Formula was not used as the value of the slope during the flood when maximum scour occured was uncertain.

CHAPTER VII

METHOD OF ANALYSIS

Analytical Considerations

The problem of scour study can be approached with Dimensional Analysis. This can aid in studying scour by giving a series of parameters; the parameters, which are dimensionless, permit limited results to be applied to cases dealing with different physical dimensions. Such is the situation with the study of scour. In this study only 13 stations were available and at each station there were several conditions that varied. Dimensional Analysis cannot be expected to produce an analytical solution to a physical problem, but it is a useful tool in formulating data which defies an analytical solution and must be solved experimentally.

The methods of Dimensional Analysis are built on the concept that an equation expressing a physical relationship between quantities should be dimensionally homogeneous. Varying sets of parameters can be found depending on which variables the investigator is considering. In the analysis interdependent variables will cancel out. Careful consideration should be given to each variable before it is included in the study. There is varying opinion as to which variables are important. The list of variables considered by this author is given in Table 8 with the symbol, units and dimensions for each given. Since three dimensions, M, L and T were involved, three repeating variables were selected;

these were taken as the velocity V_a, the depth d and density ρ.

By setting up the appropriate π parameters and solving, the following group of dimensionless parameters was obtained.

$$\left(\frac{V_a}{\sqrt{d\ g}}\ ,\ \frac{V_a\ d}{\nu}\ ,\ S\ ,\ \frac{V_a}{V_c}\ ,\ \frac{V_s}{V_f}\ ,\ \frac{d}{d_s} \right)$$

By selecting other combinations of repeating variables, it can be seen that other groups of dimensionless parameters can be obtained.

In open channel flow surface tension and compressibility are generally of minor importance. However, with free liquid surfaces the action of gravity is important. Besides the parameters of the dimensional analysis, consideration should be given the Reynolds and Froude Number.

The Froude Number is the ratio of Inertial Forces to Gravity Forces and is considered useful in open channel study. This relationship can be written as

$$\frac{F_I}{F_G} = \frac{M\ a}{M\ g} = \frac{\rho \lambda^3 \frac{V^2}{\lambda}}{\rho \lambda^3 g} = \frac{V^2}{\lambda g}\ .$$

The value of $(N_F)^{\frac{1}{2}}$ is often used in Fluid Mechanics.

In channel studies, the Reynolds Number is also important. This is because of the importance of viscosity at the low temperatures that occur during the spring breakup and flood period. The ratio of forces that establishes the value of the Reynolds Number is that of the Inertial Force divided by the Viscous Force.

This is written as

$$\frac{F_I}{F_v} = \frac{M\,a}{\mu\left(\frac{v}{l}\right)l^2} = \frac{\rho\,v\,l}{\mu} = \frac{Vl}{\nu}\,.$$

In general the values applied to the Froude Number and Reynolds Number are considered characteristic. The value of V is that of a velocity, the value of l is that of a length measurement and the value of ρ is that of a density. Used in connection with numerical analysis the values of the variables should be homogeneous to each grouping.

GRAPHICAL SOLUTION

A graphical approach was taken to investigate the relation of scour. This was done by trial and error plotting of dimensionless parameters. Also an attempt was made to see if any dimensionless parameters varied directly with the depth of scour. These ratios were plotted to find which values may influence scour and also to find which dimensionless parameters may be interdependent with each other. The plotting was done on arithmetic, semi-log and log-log paper. The figures that show a relationship, or show some interdependence, are reproduced in the list of figures and discussed in the following pages. The graphs of random configuration are not included as they are too numerous.

In Figure 15 and Figure 16, it can be seen that there is a relationship between the velocity ratio and the shear stress ratio. In both cases the velocity ratio increases as the shear stress ratio increases. In Figure 15 the fall velocity is defined as

$$V_f \quad = \quad 5.98 \sqrt{D_{50} / C_D} \quad .$$

In Figure 17, the Reynolds Number is plotted against the V_a / V_c ratio. The average velocity and depth of scour were used for the calculation of the Reynolds Number. In general the velocity ratio increases as the Reynolds Number. Because of the high degree of scatter, there is no direct relation between these two variables. Inclusion of other dimensionless variables did not reduce the scatter.

A graph of the Froude Number versus the shear stress ratio is seen in Figure 18 where the Froude Number is in terms of the average velocity and the particle size of the bed material. This graph does not include the depth of scour; it does indicate an interdependence of dimensionless quantities and also shows the importance of the Froude Number.

The plot of the depth of scour versus the critical velocity is seen in Figure 19. This graph cannot be used for scour prediction but it shows an interesting point. When the critical velocity is higher (above 1.5) the depth of scour is approximately 1 foot.

The relation between depth of scour and shear to fall velocity ratio is seen in Figure 20. The fact that many of the points are on the left is not due to clustering but because the scour depth in 9 of the 13 cases was below 2 feet. The V_s /V_f ratio is calculated from the actual shear stress and the particle size.

The relationship in Figure 20 can be given in equation form as

$$d_s = .868 + 1.88 \left(\frac{V_s}{V_f} \right) .$$

This is computed with d_s regressing onto V_s / V_f because d_s is the value to be calculated from observed data.

A similar increasing linear function is indicated in Figure 21. With d_s again as the regressing value, an equation for the depth of scour is

$$d_s = .888 + .111 \left(\frac{\tau_a}{\tau_c} \Big/ \frac{V_a}{V_c} \right) .$$

The above equations were not directly established by dimensionless analysis. However, the combinations of the variables were formed with the application of the Buckingham π Theorem.

CHAPTER VIII

CONCLUSIONS

The coefficient to the Regime equation was found to be related to the average river depth with the unit discharge defined as the total discharge divided by the width. With the data from Southwestern Ontario, the Regime equation is written as

$$d = \frac{1.42}{\frac{1.24}{q^{\frac{1}{3}}}(D_{50})^{\frac{1}{6}} + .0359} .$$

The function of Figure 2 has a coefficient of correlation of -.713 which indicates that the above Regime equation requires further study before any conclusion can be made.

With the application of dimensionless analysis, the following regression equations were determined.

$$d_s = .868 + 1.88 \left(\frac{V_s}{V_f}\right)$$

$$d_s = .888 + .111 \left(\frac{\tau_a}{\tau_c} \Big/ \frac{V_a}{V_c}\right)$$

The coefficients of correlation for these equations are .825 and .942 respectively. These values are high enough to indicate that a prediction for scour could be made within the range of the data used.

Comparison shows that where the actual velocity and boundary shear stress were respectively greater than the critical velocity and critical shear stress, scour had occured.

TABLE 1

ACTUAL DEPTH COMPARED TO DEPTH COMPUTED
BY REGIME EQUATIONS

Station	Average Depth of Scour (feet) (measured from water surface)		
	Actual	By Formula Suggested by Sanden	Findings in Project A-1 S.W. Ontario
Thames at Thamesville	18.0	40.4	17.6
Nottawasaga at Baxter	13.6	24.1	12.9
Nith at Canning	10.0	23.9	12.7
Horner at Princeton	4.6	6.48	4.65
Ausable at Springbank	14.6	28.2	14.3
Sydenham at Alvinston	2.54	4.85	3.61
Thames (S.) at Ingersoll	4.14	5.44	3.86
Nith at New Hamburg	9.43	13.8	15.6
Thames (S.) at Ealing	4.06	6.95	4.95
Saugeen near Port Elgin	14.7	23.6	15.6
Maitland below Wingham	7.43	8.9	5.95
Thames at Byron	15.4	49.8	19.2
Thames (S.) at Woodstock	4.34	5.83	4.12

TABLE 2

RESULTS OF LAURSEN FORMULA

Station	$\frac{B_1}{B_2}$	Depth of scour to Depth of Water Ratio: Laurson Formula	Actual
Thames at Thamesville	1.20	.124	.167
Nottawasaga at Baxter	1.80	.415	.458
Nith at Canning	1.21	.119	.127
Horner at Princeton	1.67	.354	.396
Ausable at Springbank	1.25	.154	.146
Sydenham at Alvinston	1.80	.430	.633
Thames (S.) at Ingersoll	1.25	.157	.360
Nith at New Hamburg	1.20	.119	.108
Thames (S.) at Ealing	1.38	.209	.268
Saugeen near Port Elgin	1.17	.097	.089
Maitland below Wingham	1.22	.124	.191
Thames at Byron	1.23	.154	.260
Thames (S.) at Woodstock	1.75	.391	.389

TABLE 3

MAP IDENTIFICATION

River and Location	Hydrology Reference Number	Number on Figure 6
Thames at Thamesville	2 G E_3	1
Nottawasaga at Baxter	2 E D_3	2
Nith at Canning	2 G A_{10}	3
Horner at Princeton	2 G B_6	4
Ausable at Springbank	2 F F_2	5
Sydenham at Alvinston	2 G G_6	6
Thames at Ingersoll	2 G D_{16}	7
Nith at New Hamburg	2 G A_{18}	8
Thames (S.) at Ealing	2 G D_1	9
Saugeen near Port Elgin	2 F C_1	10
Maitland below Wingham	2 F E_2	11
Thames at Byron	2 G E_2	12
Thames (S.) at Woodstock	2 G D_{12}	13

TABLE 4

TABULATION OF PARTICLE SIZE

Station	Particle Size In Millimeters Percent Retained		
	25	35	50
Thames at Thamesville	.22	.18	.14
Nottawasaga at Baxter	.27	.25	.22
Nith at Canning	13.0	9.00	4.60
Horner at Princeton	.45	.37	.35
Ausable at Springbank	.50	.40	.20
Sydenham at Alvinston	.44	.32	.21
Thames at Ingersoll	9.40	6.50	3.20
Nith at New Hamburg	4.85	3.32	1.39
Thames (S.) at Ealing	5.8	1.8	.72
Saugeen near Port Elgin	17.0	12.7	7.1
Maitland below Wingham	4.4	2.3	.95
Thames at Byron	.20e	.11e	.055e
Thames at Woodstock	.45	.40	.34

TABLE 5

CHANNEL SLOPE, SPECIFIC GRAVITY and
BOUNDARY ROUGHNESS COEFFICIENT AT EACH STATION

Station	Slope ft./ft.	Specific Gravity	Boundary Roughness Coefficient
Thames at Thamesville	.00048	2.74	.0100
Nottawasaga at Baxter	.00066	2.69	.0104
Nith at Canning	.00101	2.68	.0204
Horner at Princeton	.000169	2.72	.0120
Ausable at Springbank	.000858	2.64	.0115
Sydenham at Alvinston	.000194	2.54	.0105
Thames at Ingersoll	.00137	2.68	.0181
Nith at New Hamburg	.000417	2.65	.0169
Thames at Ealing	.000740	2.71	.0159
Saugeen near Port Elgin	.000192	2.68	.0198
Maitland below Wingham	.000129	2.61	.0169
Thames at Byron	.000350	2.66 e	.00932
Thames at Woodstock	.000174	2.60	.0117

TABLE 6

SLOPE EXTRAPOLATION

Nottawasaga River at Baxter

	Date	Flow cfs	Stage feet	Slope ft/ft
By Water Resources Branch	4-9-64	883	641.02	.00032
Project (A-1)	5-19-64	164	637.86	.00017
Slope Extrapolation	4-4-60	6,177	650.64	.00066

Thames River at Thamesville

	Date	Flow cfs	Stage feet	Slope ft/ft
By Water Resources Branch	4-8-64	7,720	598.18	.00040
Project (A-1)	5-4-64	864.	586.84	.000262
Slope Extrapolation	3-28-63	20,700	609.95	.00048

TABLE 7

COMPARISON OF ACTUAL SHEAR STRESS TO CRITICAL SHEAR STRESS

Station	Actual Shear Stress	Critical Shear Stress
Thames near Thamesville	.214	.00282
Nottawasaga at Baxter	.224	.00375
Nith at Canning	.417	.1073
Horner at Princeton	.045	.00438
Ausable at Springbank	.207	.00438
Sydenham at Alvinston	.060	.00401
Thames at Ingersoll	.070	.0463
Nith at New Hamburg	.136	.0367
Thames at Ealing	.255	.0123
Saugeen near Port Elgin	.479	.1570
Maitland below Wingham	.099	.0295
Thames at Byron	.227	.00159
Thames at Woodstock	.022	.00610

TABLE 8

QUANTITIES CONSIDERED FOR DIMENSIONAL ANALYSIS

Quantity	Symbol	Units ft.lbs.sec.	Dimensions M - L - T
Shear Stress	τ_a	lbs/sq.ft.	$M^1 L^{-1} T^{-2}$
Critical Shear Stress	τ_c	lbs/sq.ft.	$M^1 L^{-1} T^{-2}$
Depth	d	ft.	L^1
Depth of scour	d_s	ft.	L^1
Density	ρ	$\frac{\text{slugs}}{\text{cu.ft.}}$	$M^1 L^{-3}$
Gravitational Acceleration	g	ft/sec^2.	$L^1 T^{-2}$
Kinematic Viscosity	ν	ft^2/sec.	$L^2 T^{-1}$
Particle Size	D	ft.	L^1
Average Velocity	v_a	ft/sec.	$L^1 T^{-1}$
Critical Velocity	v_c	ft/sec.	$L^1 T^{-1}$
Fall Velocity (particles)	v_f	ft/sec.	$L^1 T^{-1}$
Shear Velocity	v_s	ft/sec.	$L^1 T^{-1}$
Slope	S		

Thames River at Thamesville. Station Number 2GE 3. March 28, 1963.

Station	Velocity .2 depth	Velocity .8 depth	Velocity Average	Depth d	Froude No. $N_F = \frac{V}{\sqrt{gd}}$	Specific Energy $E = d + \frac{V^2}{2g}$	Reynolds No. $N_R = \frac{V d}{\nu}$ X 10^{-5}	Shear Stress $\tau_a = 1.94\left(\frac{V_1 - V_2}{5.75 \log \frac{y_1}{y_2}}\right)^2$	Critical Velocity at bottom (19) $V_c = 8.45\sqrt{\frac{\gamma_s - \gamma_w}{\gamma_w}}\left(\frac{d}{D_{50}}\right)^{\frac{1}{6}}\sqrt{D_{50}}$	Shear Velocity $V_s = \sqrt{\frac{\tau_a}{\rho}}$	Fall Velocity $V_f = 5.98\sqrt{\frac{D_{50}}{C_D}}$	V_s/V_f
(feet)	(fps)	(fps)	(fps)	(feet)		(feet)		(lbs./sq.ft.)	(fps)	(fps)	(fps)	
40	1.41	.59	1.00	12.6	.05	12.62	6.5	.11	1.31	.238	.287	.830
50	1.50	.78	1.14	16.0	.06	16.02	9.5	.08	1.37	.203	.287	.707
60	2.83	1.44	2.14	18.3	.09	18.37	20.5	.31	1.40	.399	.287	1.39
70	3.69	1.35	2.52	22.1	.10	22.21	28.8	.89	1.44	.677	.287	2.36
80	4.40	3.39	3.90	26.2	.14	26.44	53.0	.17	1.49	.296	.287	1.03
90	4.43	4.25	4.34	27.0	.15	27.29	60.7	.01	1.50	.072	.287	.251
100	5.42	4.89	5.16	27.7	.17	28.11	73.8	.05	1.50	.160	.287	.558
110	6.03	3.66	4.84	29.0	.16	29.45	72.7	.91	1.51	.684	.287	2.38
120	6.36	5.23	5.80	29.3	.19	29.76	88.0	.21	1.51	.328	.287	1.14
130	6.42	5.63	6.02	30.7	.19	31.26	95.5	.10	1.52	.227	.287	.791
140	7.01	5.49	6.25	30.0	.20	30.61	97.2	.37	1.52	.436	.287	1.520
150	6.76	5.63	6.20	28.5	.21	29.09	91.5	.21	1.50	.329	.287	1.15
160	6.80	5.63	6.22	29.0	.20	29.60	93.5	.22	1.51	.337	.287	1.18
170	6.68	5.42	6.05	28.7	.20	29.27	90.0	.26	1.51	.366	.287	1.28
180	6.28	5.36	5.82	27.4	.20	27.92	82.5	.14	1.50	.268	.287	.935
190	5.42	4.74	5.08	25.0	.18	25.40	65.8	.07	1.48	.190	.287	.662
200	4.35	3.85	4.10	21.7	.16	21.96	46.2	.04	1.44	.144	.287	.502
207	3.04	1.53	2.28	21.7	.09	21.78	25.6	.37	1.44	.437	.287	1.520
212	1.83	1.35	1.59	18.1	.07	18.14	14.9	.04	1.40	.144	.287	.502
220	1.35	.75	1.05	14.3	.05	14.32	7.8	.06	1.34	.176	.287	.614
230	.75	.28	.52	10.0	.03	10.00	1.5	.04	1.27	.144	.287	.502
240	.77	.28	.38	6.2	.03	6.20	1.2	.04	1.17	.144	.287	.502
Average Values			4.29	18.4	.133		51.2	.214	1.44	$V_s = \sqrt{\frac{\tau_{av.}}{\rho}} = .291$	.287	1.16

TABLE 9

Nottawasaga River at Baxter. Station Number 2ED 3. April 4, 1960.

Station	Velocity .2 depth	Velocity .8 depth	Velocity Average	Depth d	Froude No. $N_F = \frac{V}{\sqrt{gd}}$	Specific Energy $E = d + \frac{V^2}{2g}$	Reynolds No. $N_R = \frac{V d}{\nu}$ X 10^{-5}	Shear Stress $\tau_o = 1.94\left(\frac{V_1 - V_2}{5.75 \log \frac{y_1}{y_2}}\right)^2$	Critical Velocity at bottom (19) $V_c = 8.45\sqrt{\frac{\gamma_s - \gamma_w}{\gamma_w}}\left(\frac{d}{D_{50}}\right)^{\frac{1}{6}}\sqrt{D_{50}}$	Shear Velocity $V_s = \sqrt{\frac{\tau_o}{\rho}}$	Fall Velocity $V_f = 5.98\sqrt{\frac{D_{50}}{C_D}}$	V_s/V_f
(feet)	(fps)	(fps)	(fps)	(feet)		(feet)		(lbs./sq.ft.)	(fps)	(fps)	(fps)	
65	2.33e	1.66e	2.00	11.5	.10	11.56	15.0	.07	1.45	.190	.359	.529
70	2.78e	1.99e	2.39	14.5	.11	14.59	22.6	.10	1.51	.227	.359	.633
80	3.73e	2.66e	3.20	13.7	.15	13.86	28.5	.18	1.50	.305	.359	.840
90	4.44e	3.17e	3.81	20.1	.14	20.32	49.8	.26	1.59	.366	.359	1.02
100	5.78e	4.13e	4.96	21.8	.19	22.18	70.4	.44	1.62	.475	.359	1.32
110	6.53e	4.66e	5.60	22.2	.21	22.69	80.8	.57	1.62	.542	.359	1.51
120	5.14e	3.67e	4.42	21.1	.17	21.40	60.6	.35	1.61	.425	.359	1.18
130	2.34e	1.67e	2.01	20.9	.07	20.96	27.3	.07	1.61	.190	.359	.529
131	1.95e	1.39e	1.67	12.5	.10	12.54	13.6	.05	1.47	.161	.359	.448
135	4.09e	2.92e	3.51	13.8	.17	13.99	31.5	.22	1.47	.336	.359	.935
140	4.53e	3.22e	3.87	13.5	.19	13.73	34.0	.27	1.50	.373	.359	1.04
150	1.28e	.92e	1.10	13.5	.06	13.51	9.65	.02	1.50	.102	.359	.284
155	.91e	.65e	.78	10.1	.04	10.11	5.13	.01	1.47	.072	.359	.200
Average Values			3.47	14.0	.141		35.17	.224	1.55	$V_s = \sqrt{\frac{\tau_{ave.}}{\rho}} = .340$	.359	.949

TABLE 10

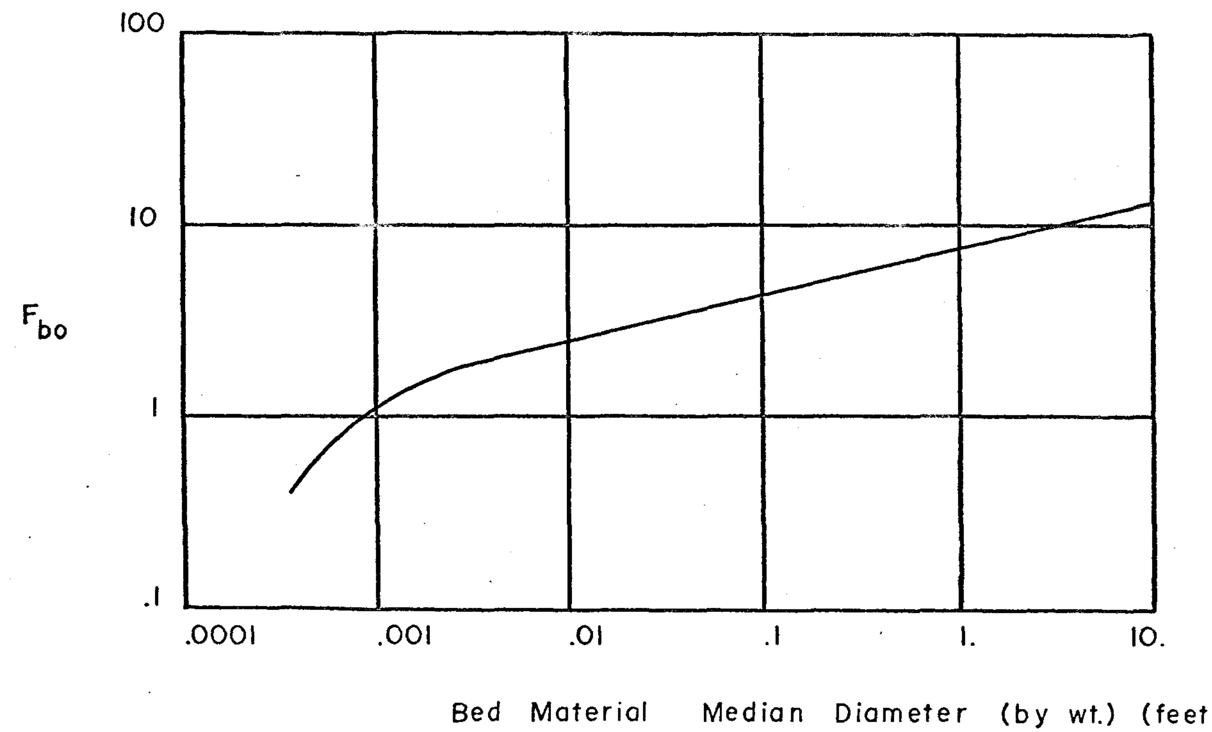


FIGURE I Graph of "zero bed-factor"

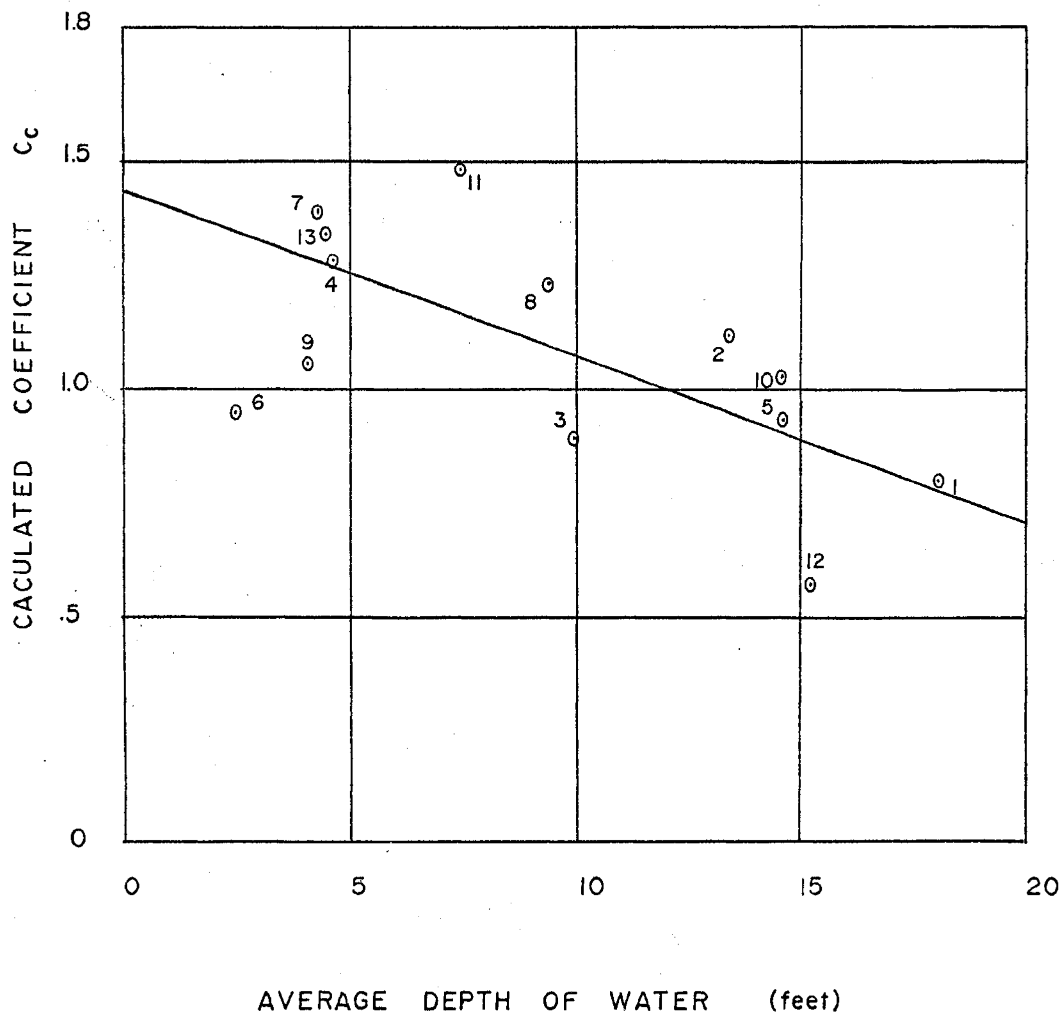


FIGURE 2 Calculated coefficient of "Blench" equation versus the average depth of water.

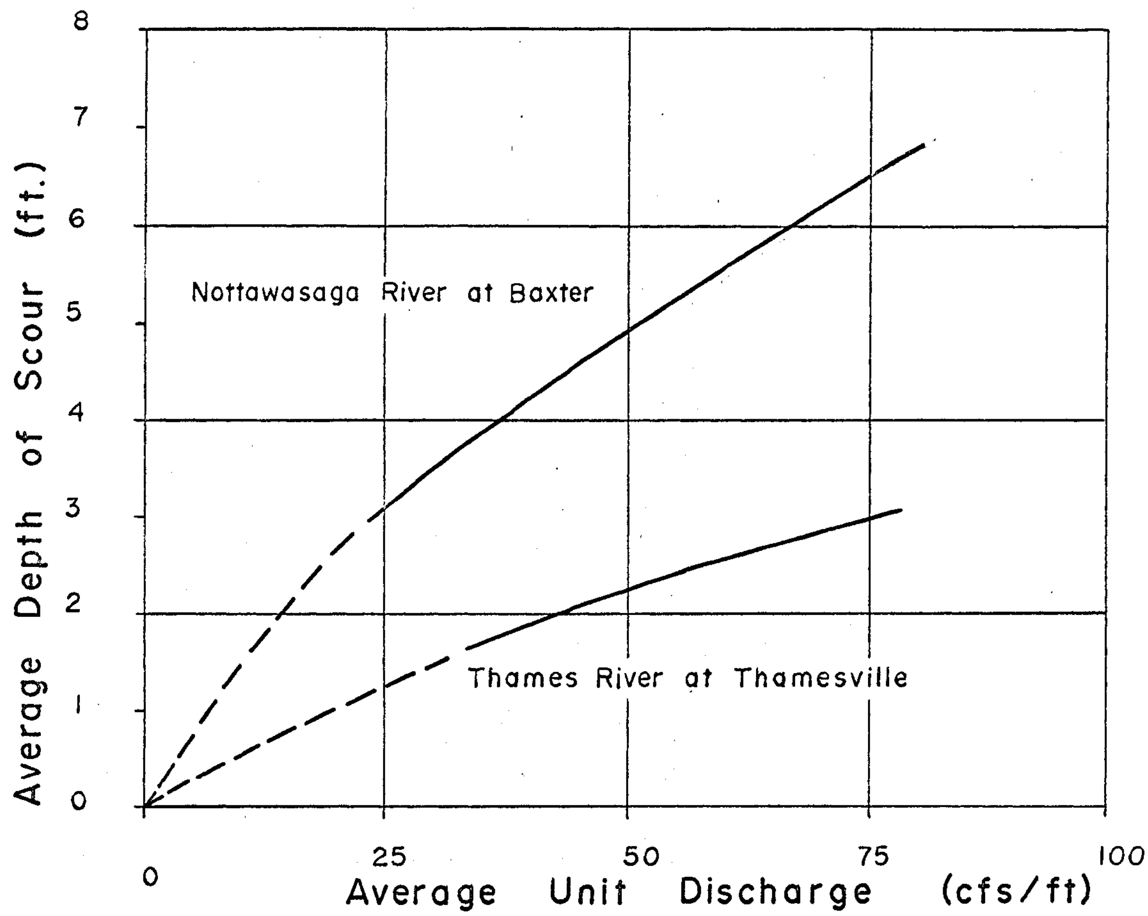


Figure 3 Relation of depth of scour to unit discharge as expressed by N. Ahmid.

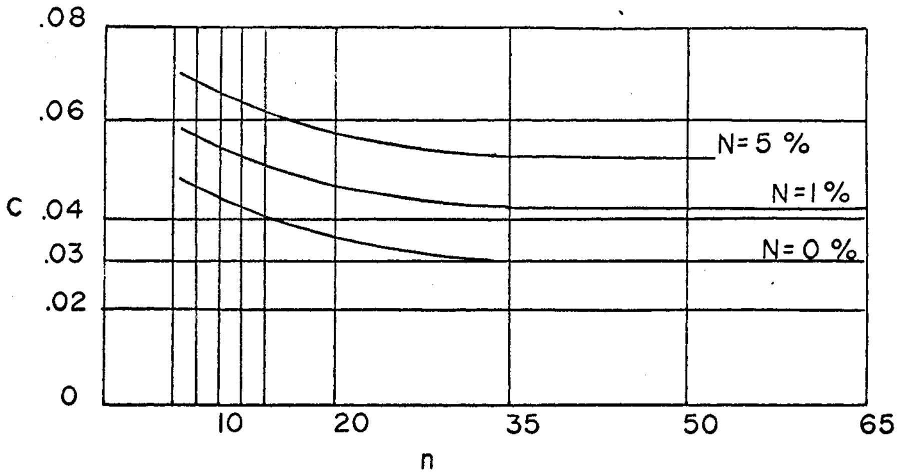


FIGURE 4 a Coefficient "C" as a function of degree of erosion "N" and inclination 1:n of bottom slope, taken from S. Andersson.

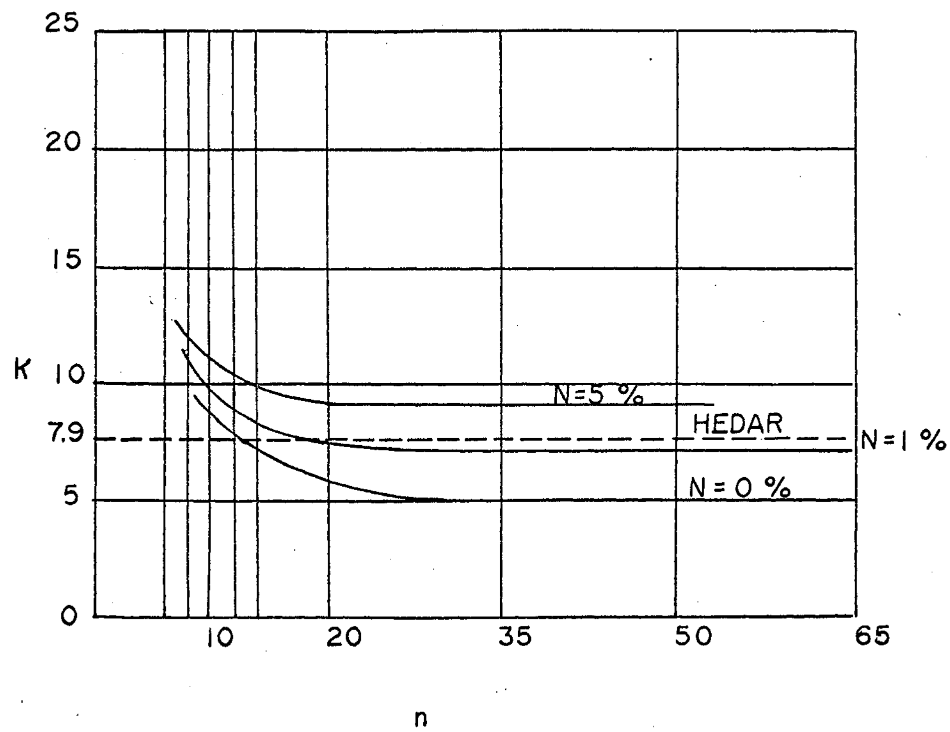


FIGURE 4 b Cofficient "K" as a function of degree of erosion "N" and inclination 1:n of bottom slope, taken from S. Andersson.

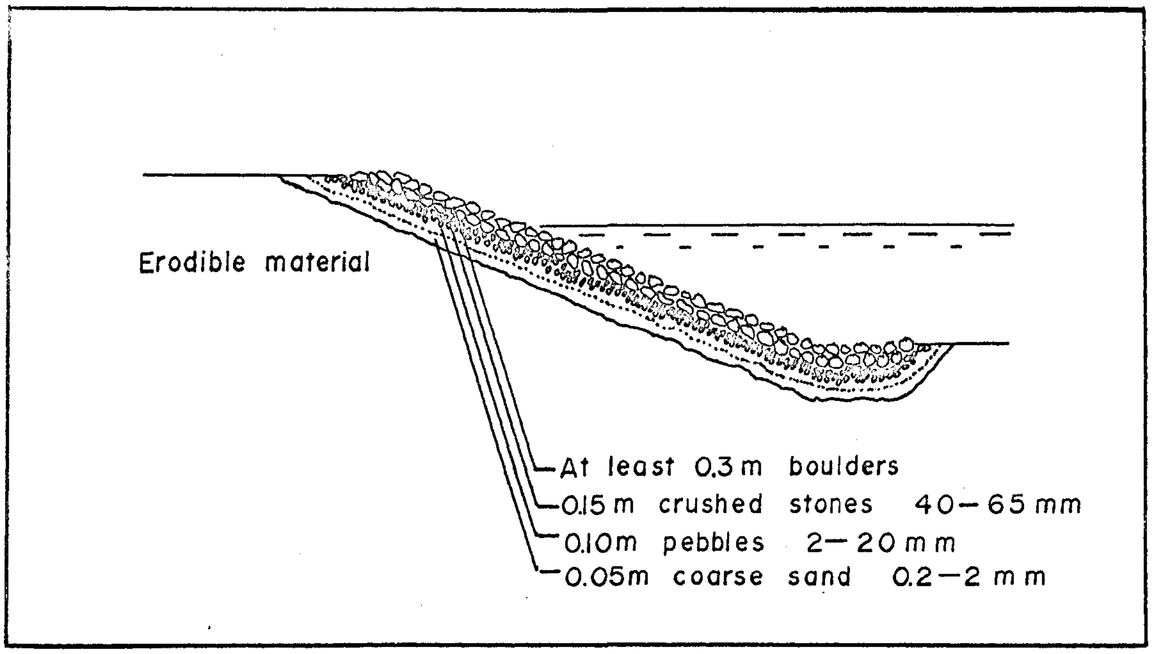


FIGURE 5 a Traditional filter of classified material.

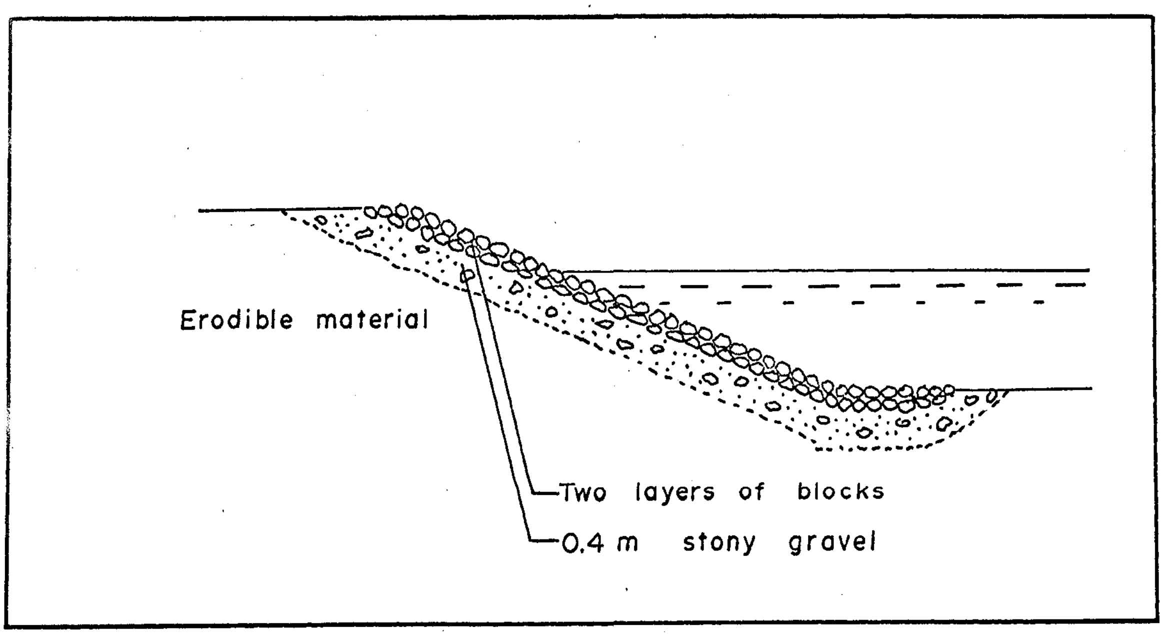


FIGURE 5 b Filter of well graded material with armour layer of stones.

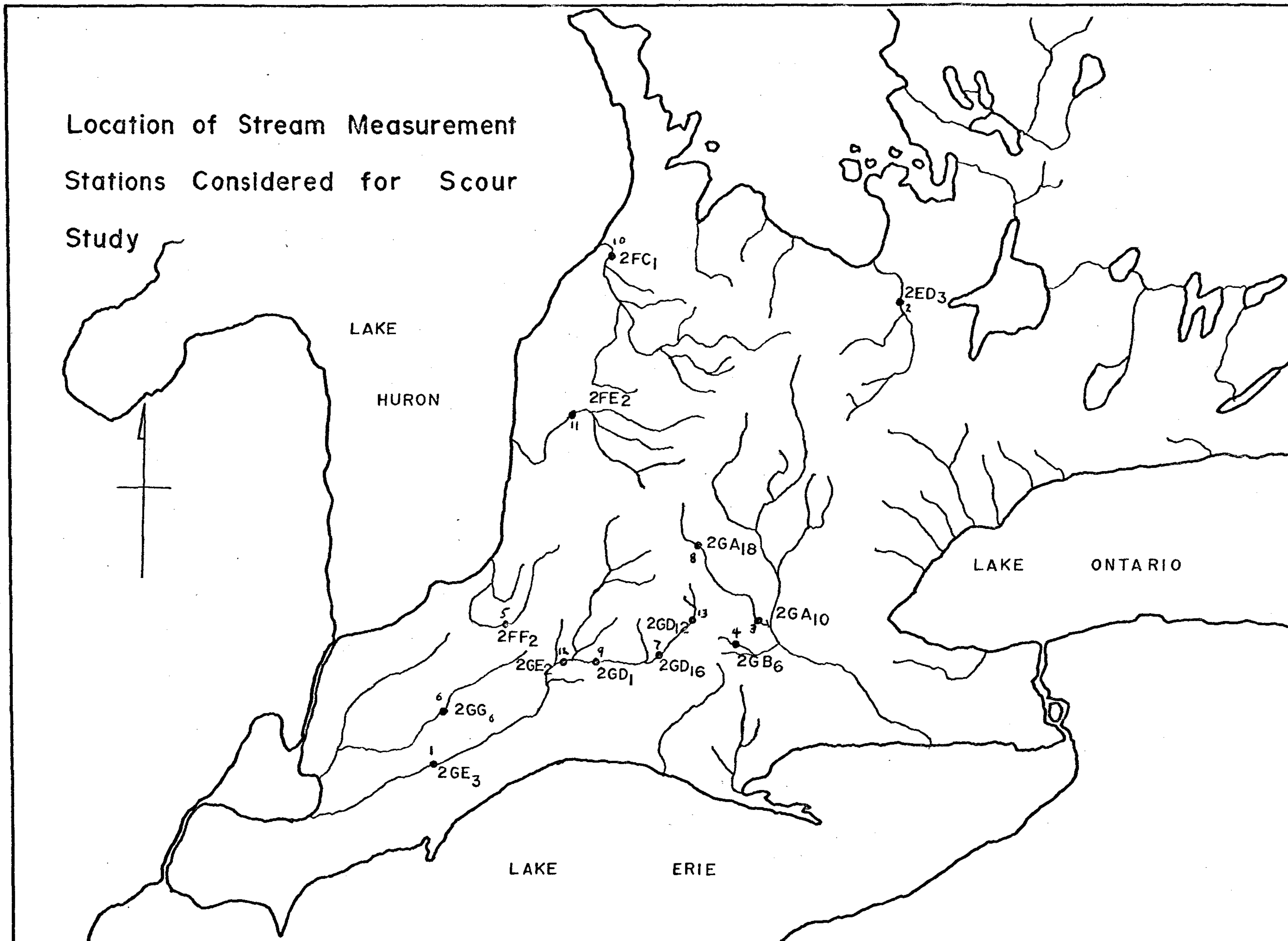

Location of Stream Measurement
Stations Considered for Scour
Study
LAKE
HURON
LAKE ONTARIO
LAKE ERIE
2FC1
10
2ED3
2
2FE2
11
2GA18
8
2GA10
3
2GD12
13
2FF2
5
2GE2
12
2GD1
9
2GD16
7
2GB6
4
2GG6
6
2GE3
1

FIGURE 6

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

THAMES RIVER AT THAMESVILLE   Sample taken 18 feet from left pier at a depth of 6 inches.

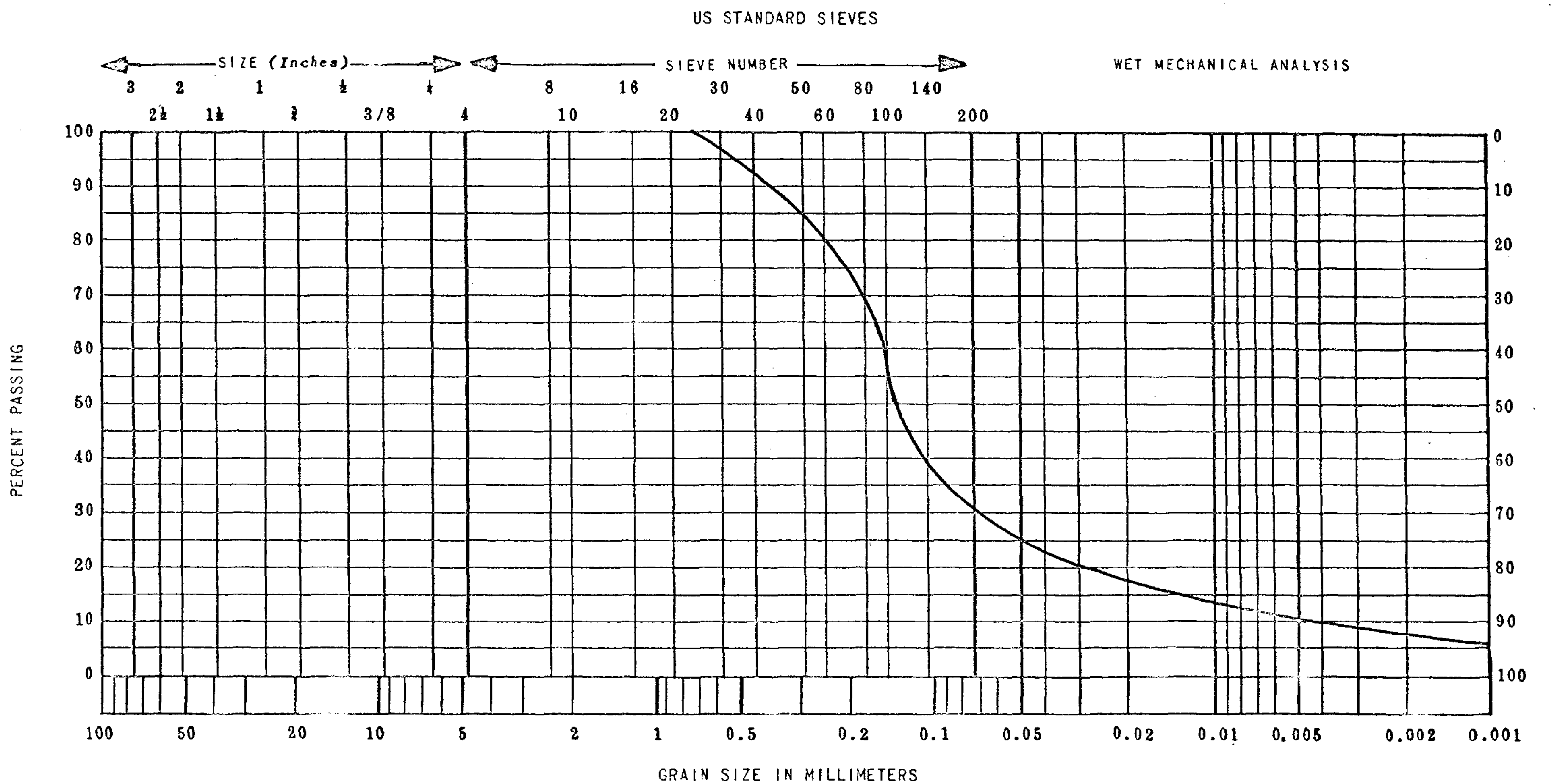


FIGURE 7

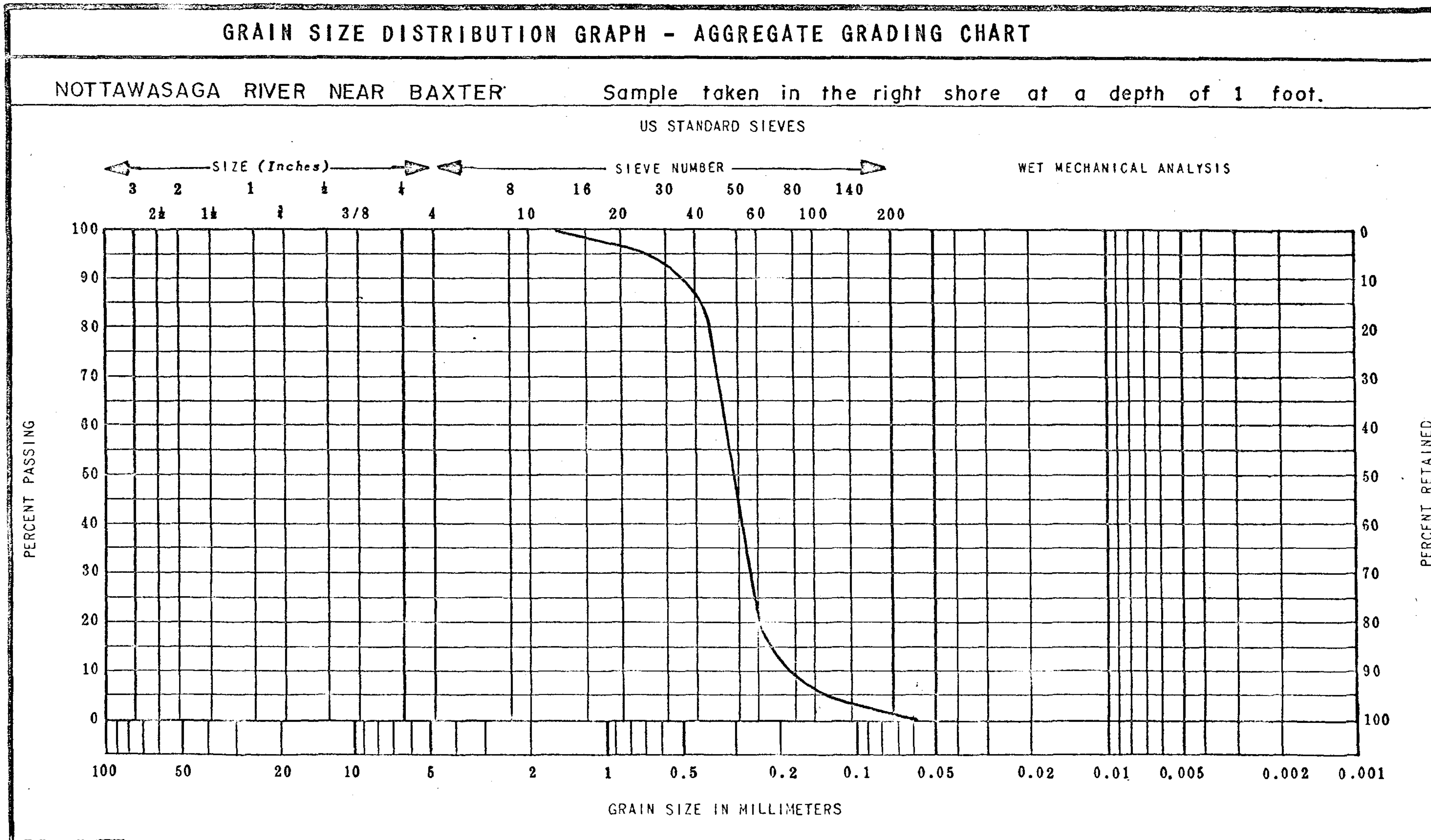

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART
NOTTAWASAGA RIVER NEAR BAXTER
Sample taken in the right shore at a depth of 1 foot.
US STANDARD SIEVES
SIZE (Inches)
SIEVE NUMBER
WET MECHANICAL ANALYSIS
3 2 1 ½ ¼
2½ 1½ ¾ 3/8 4
8 16 30 50 80 140
10 20 40 60 100 200
PERCENT PASSING
100 90 80 70 60 50 40 30 20 10 0
PERCENT RETAINED
0 10 20 30 40 50 60 70 80 90 100
100 50 20 10 5 2 1 0.5 0.2 0.1 0.05 0.02 0.01 0.005 0.002 0.001
GRAIN SIZE IN MILLIMETERS

FIGURE 8

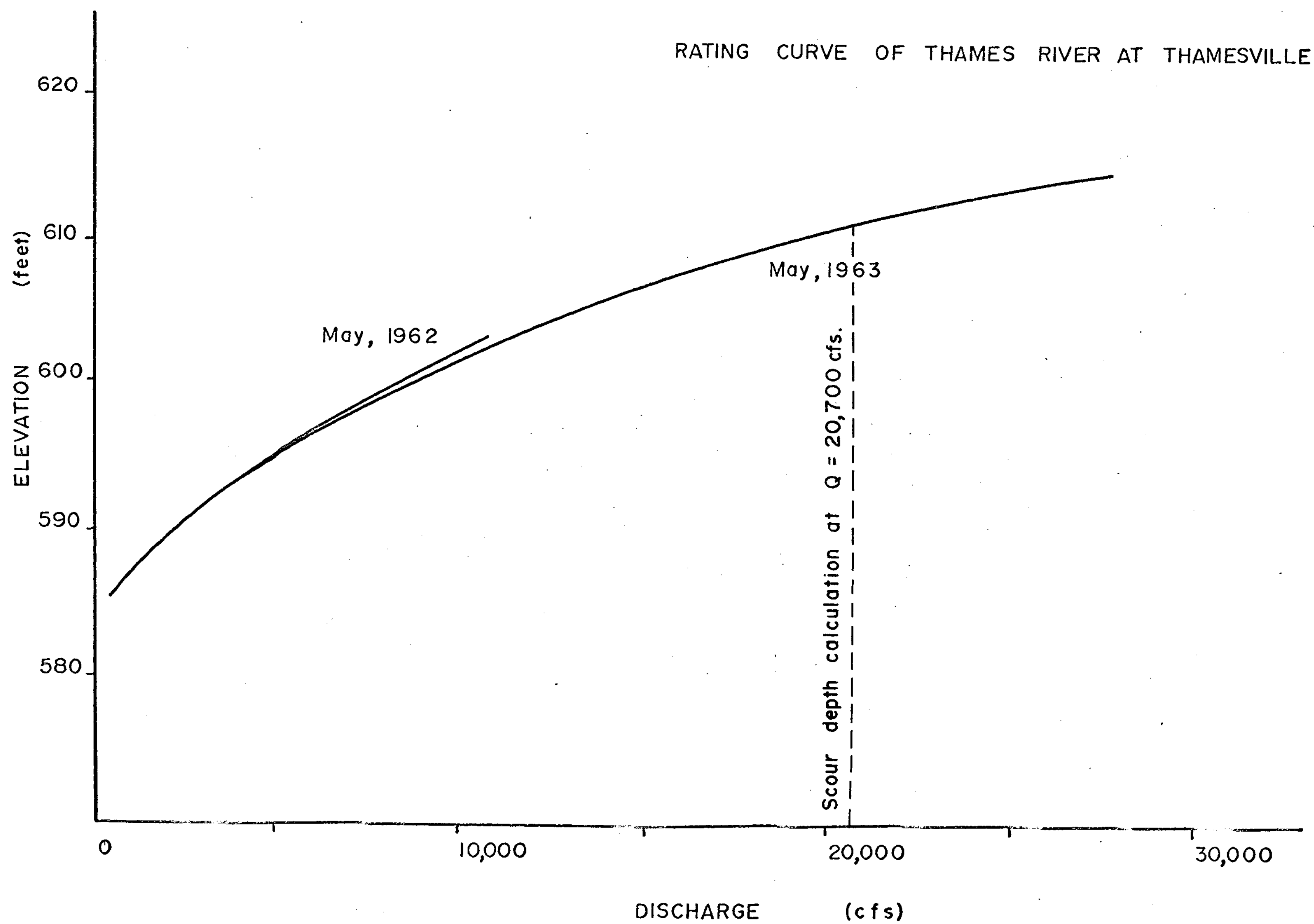

RATING CURVE OF THAMES RIVER AT THAMESVILLE
ELEVATION (feet)
620
610
600
590
580
May, 1962
May, 1963
Scour depth calculation at Q = 20,700 cfs.
0
10,000
20,000
30,000
DISCHARGE (cfs)

FIGURE 9

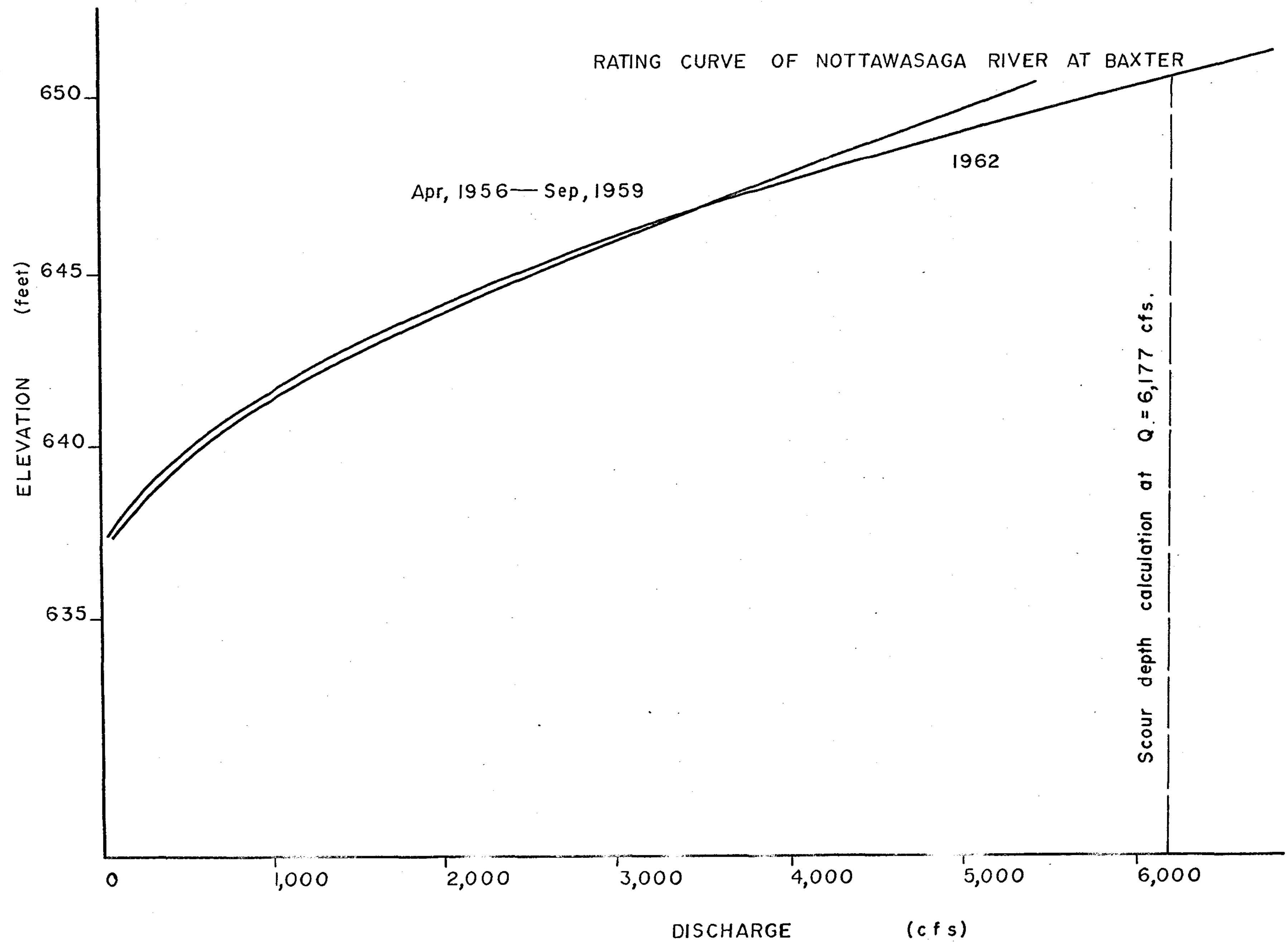

RATING CURVE OF NOTTAWASAGA RIVER AT BAXTER
1962
Apr, 1956 — Sep, 1959
Scour depth calculation at Q = 6,177 cfs.
ELEVATION (feet)
650
645
640
635
DISCHARGE (cfs)
0
1,000
2,000
3,000
4,000
5,000
6,000

FIGURE 10

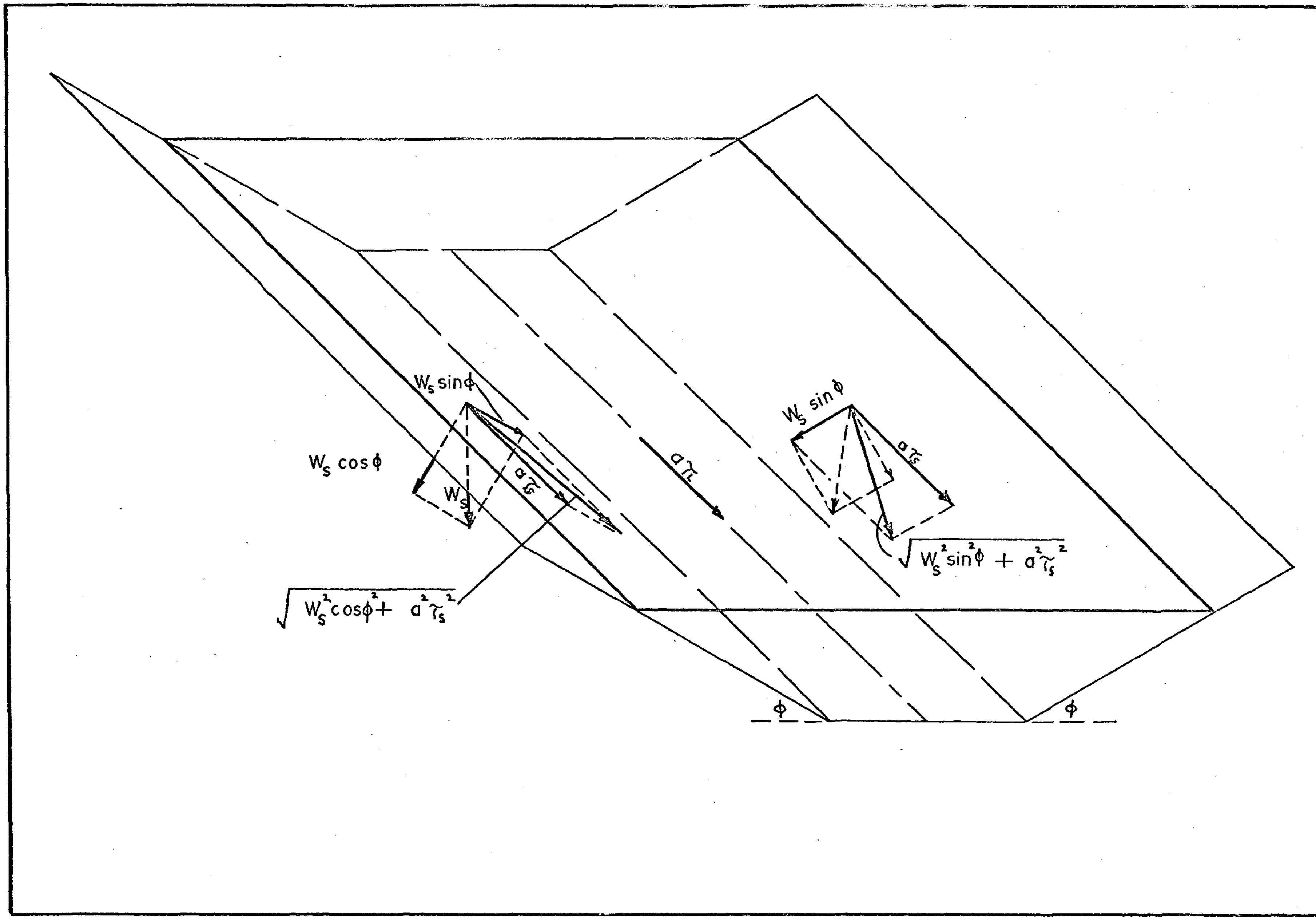


FIGURE 11 Analysis of forces acting on a particle resting on the surface of a channel bed. (Ref. 5)

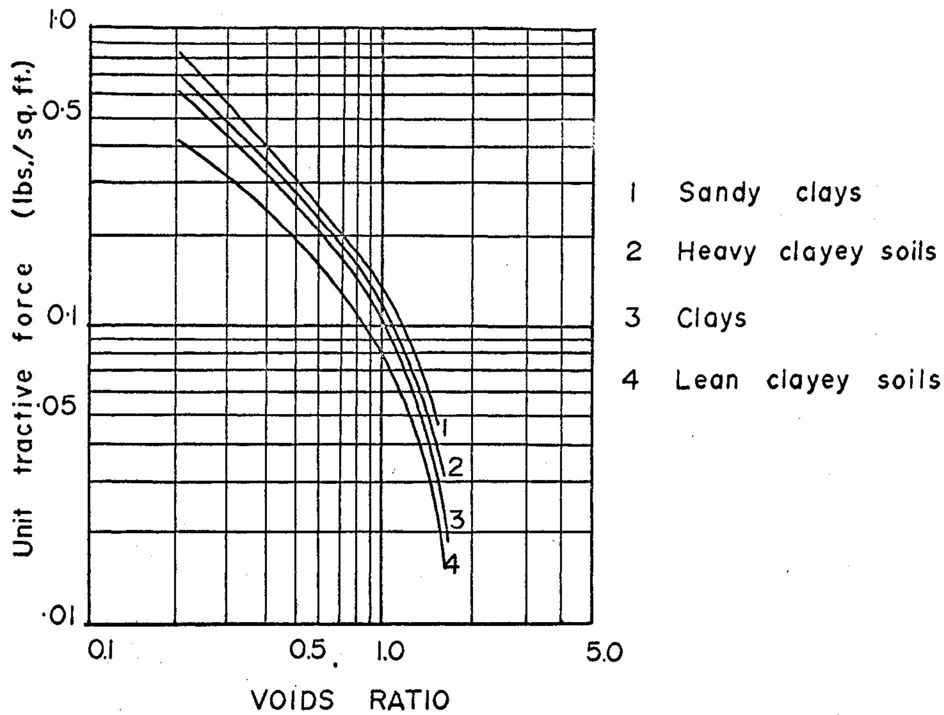


FIGURE 12(a) Permissible unit tractive force for canals in cohesive material. (from U.S.S.R. data)

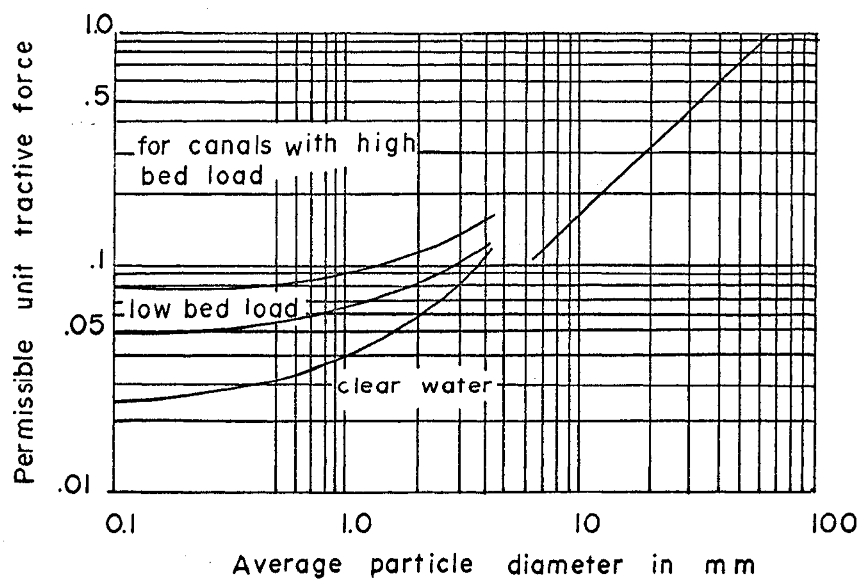


FIGURE 12 (b) Recommended permissible unit tractive force for canals in noncohesive material. (U.S. Bureau of Reclamation)

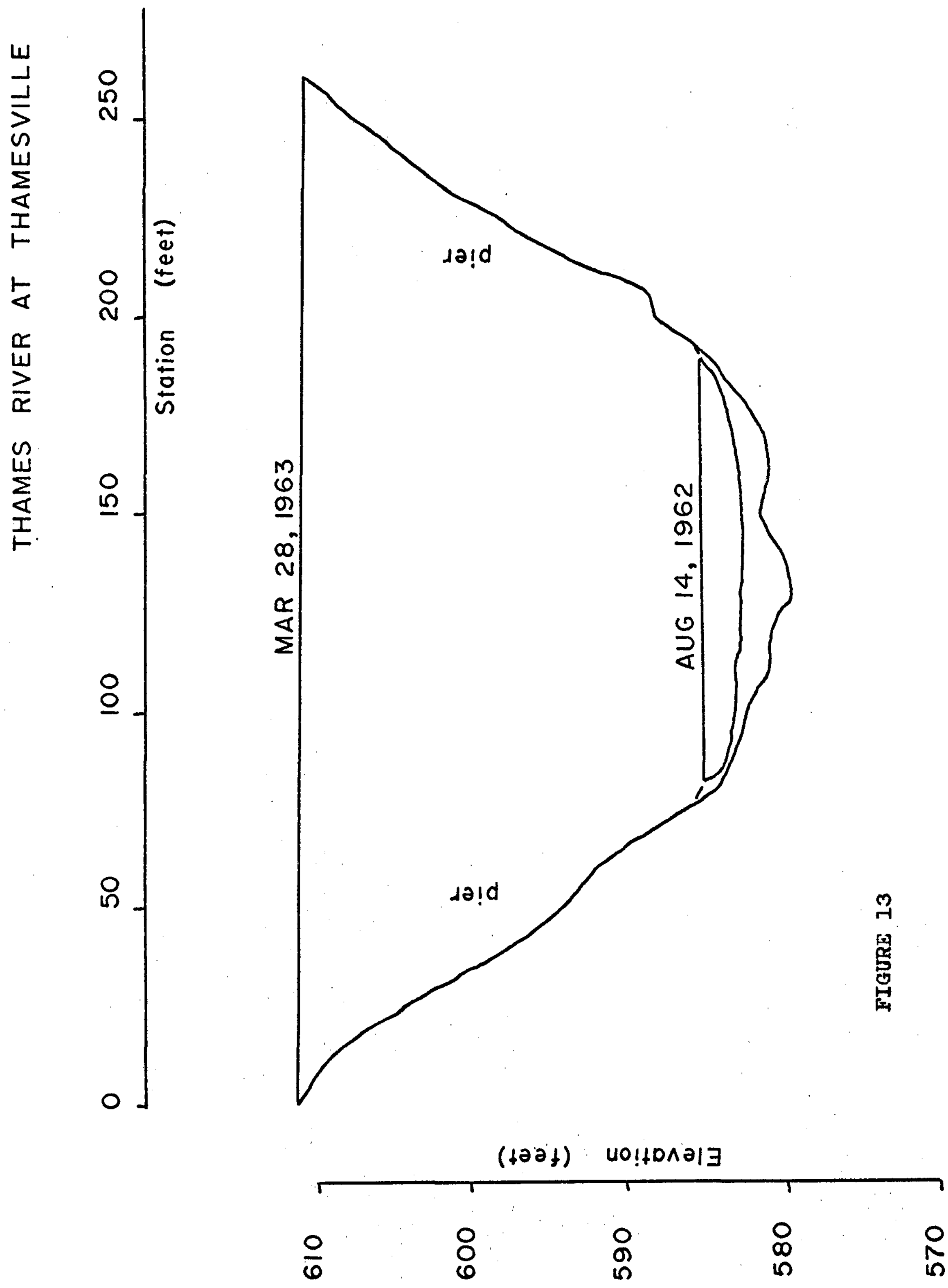

THAMES RIVER AT THAMESVILLE
Station (feet)
0
50
100
150
200
250
MAR 28, 1963
AUG 14, 1962
pier
pier
Elevation (feet)
610
600
590
580
570

FIGURE 13

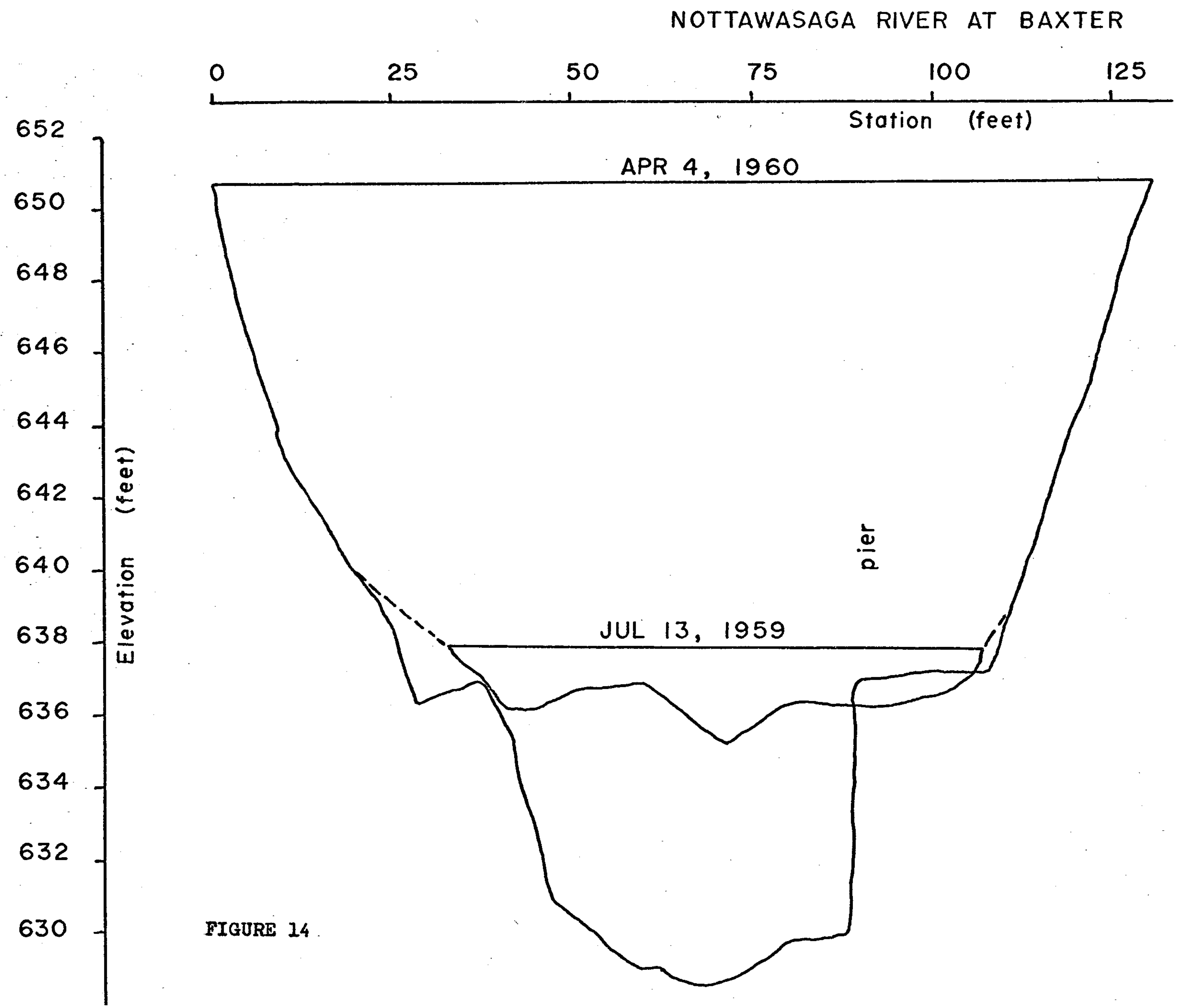

NOTTAWASAGA RIVER AT BAXTER
0
25
50
75
100
125
Station (feet)
APR 4, 1960
JUL 13, 1959
pier
652
650
648
646
644
642
640
638
636
634
632
630
Elevation (feet)

FIGURE 14

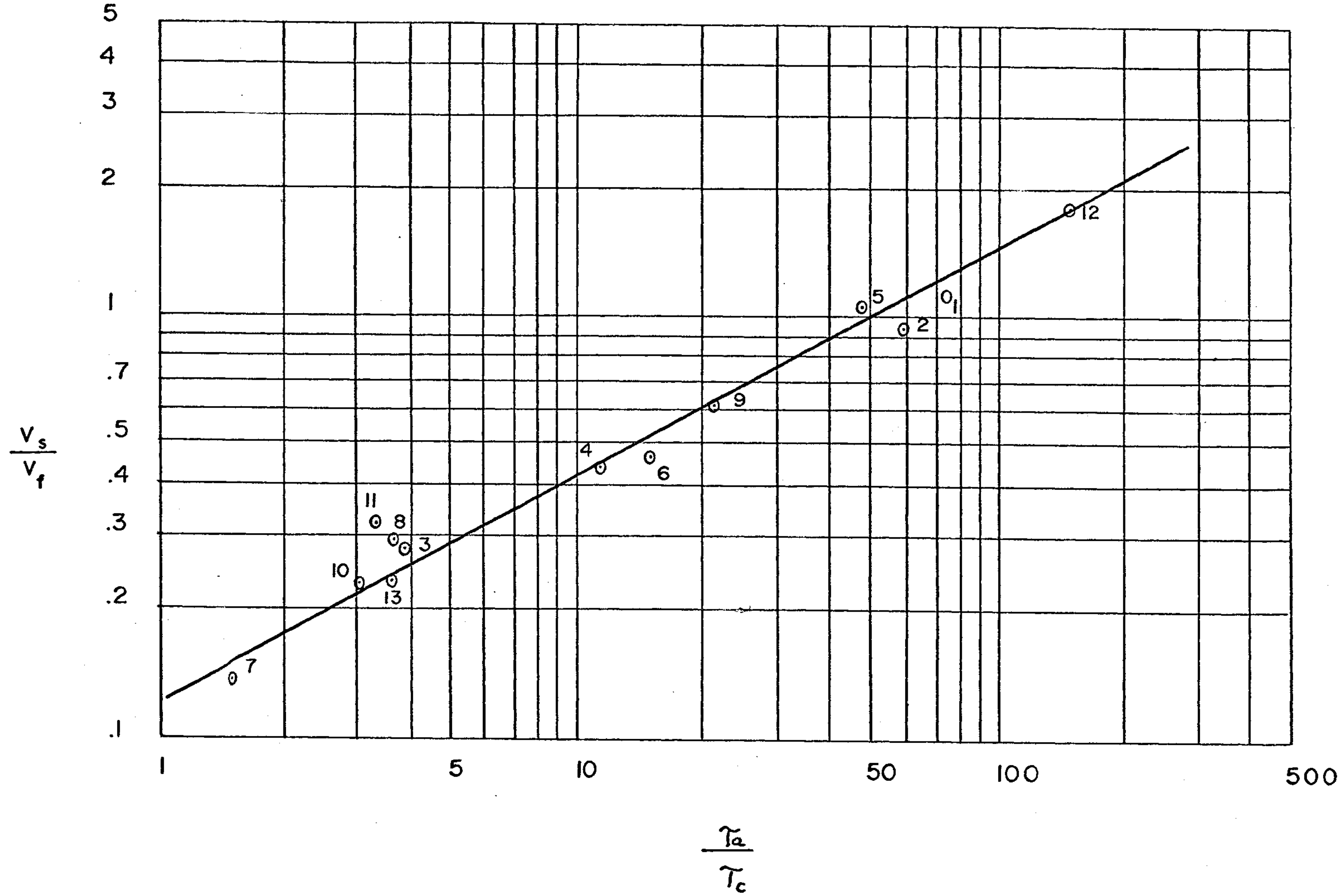


FIGURE 15 Plot of the shear to fall velocity ratio against the average to critical shear stress ratio.

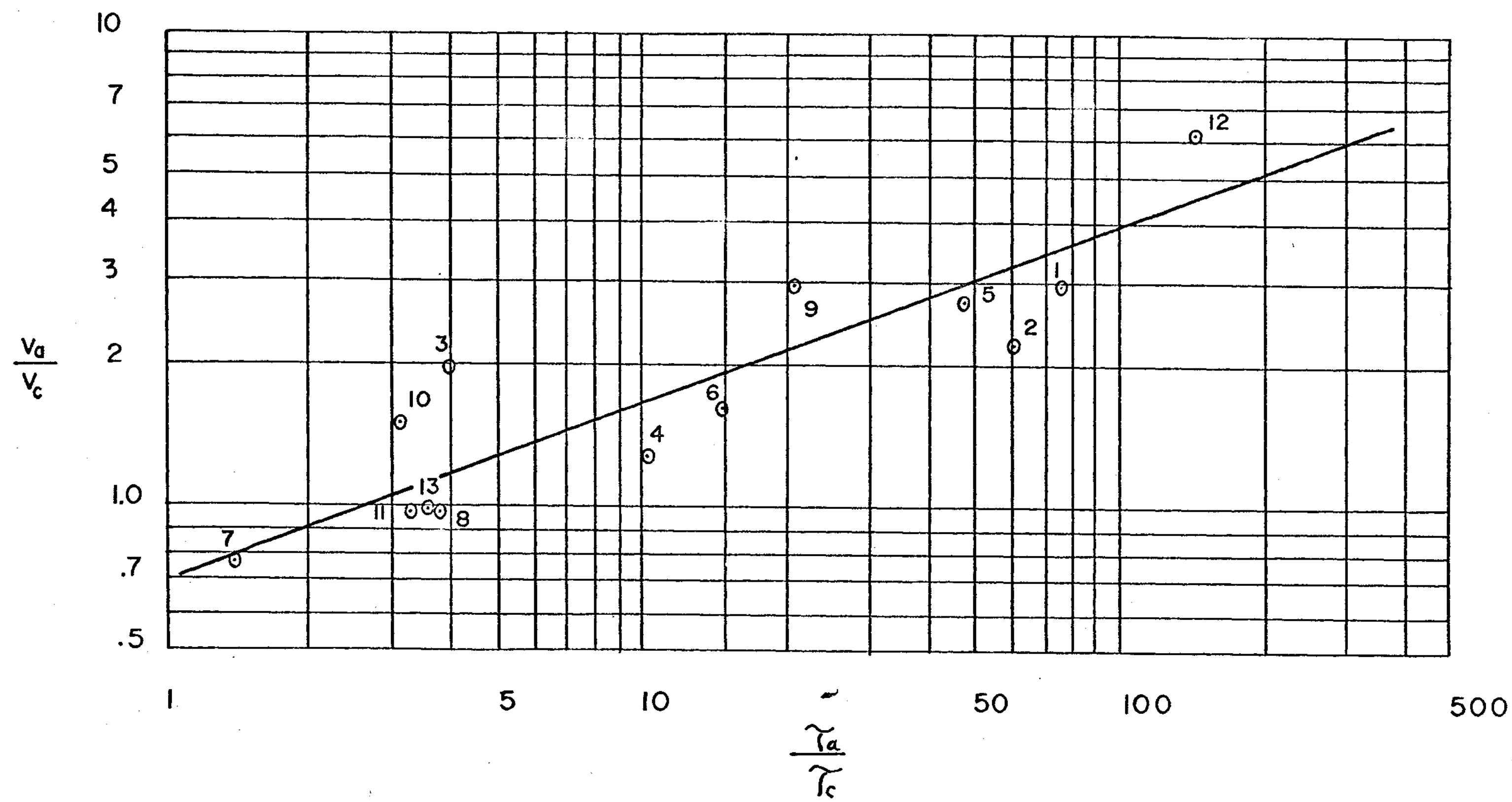
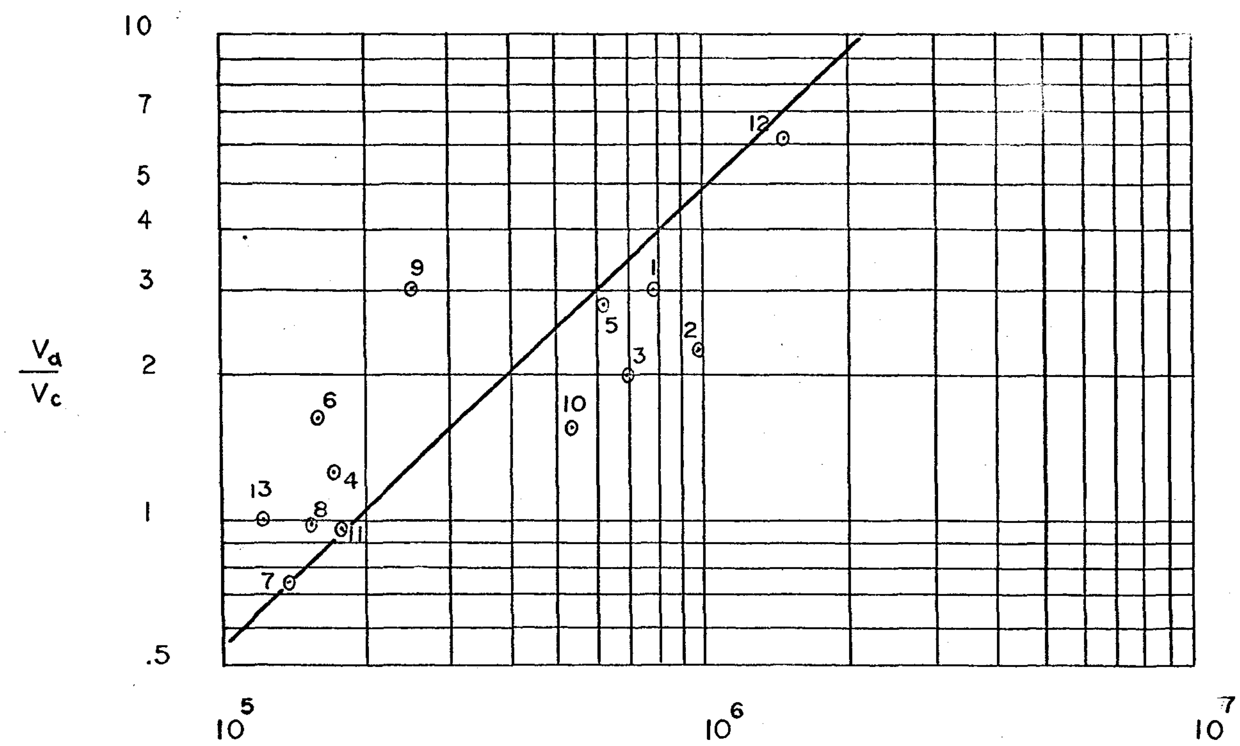


FIGURE 16 Plot of the average to critical velocity ratio against the average to critical shear stress ratio.



$$N_R = \frac{V_a d_s}{\nu}$$

FIGURE 17 Plot of Reynolds Number against the actual to critical velocity ratio.

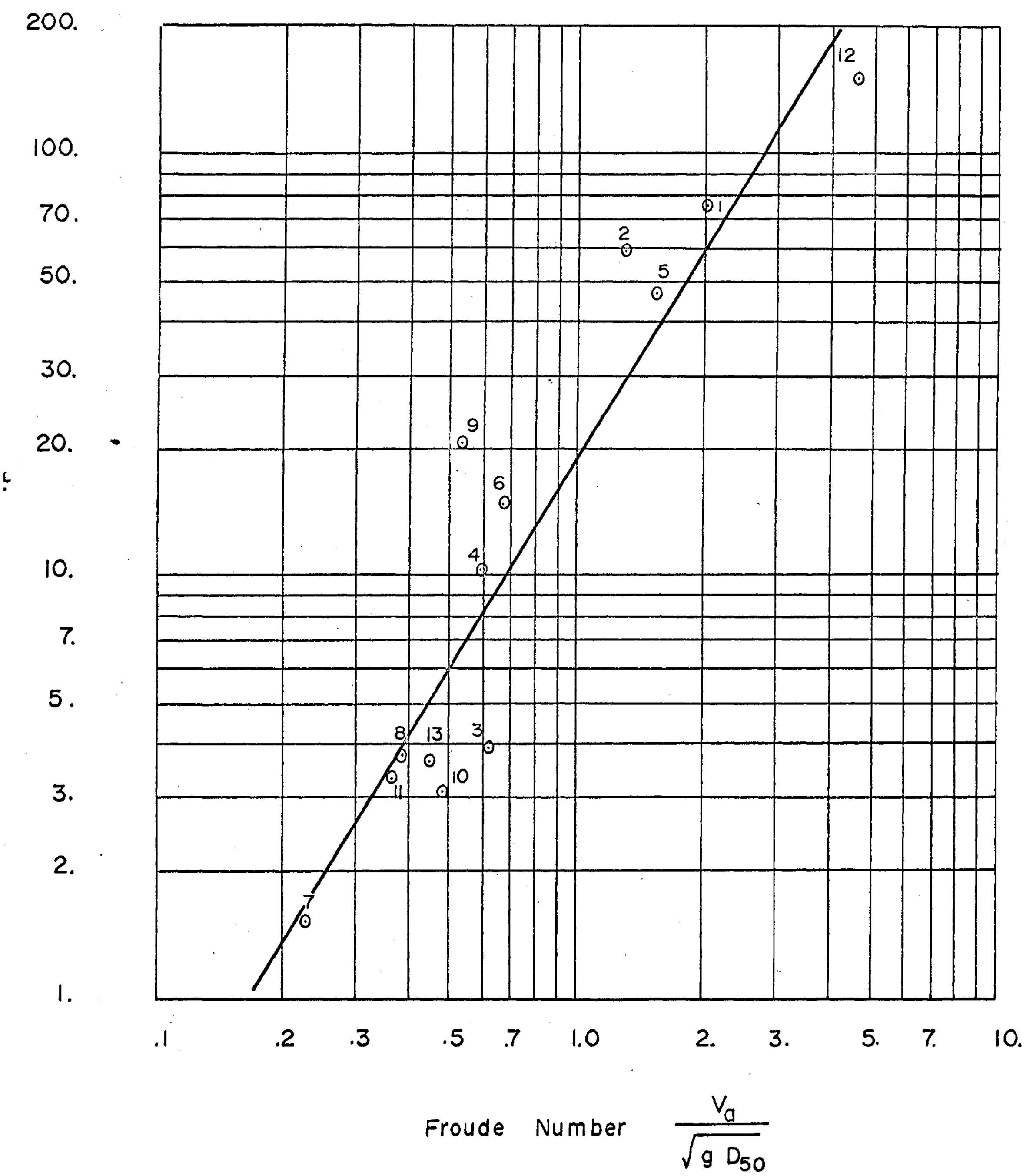


FIGURE 18 Graph showing relation between the shear stress ratio and a defined Froude Number.

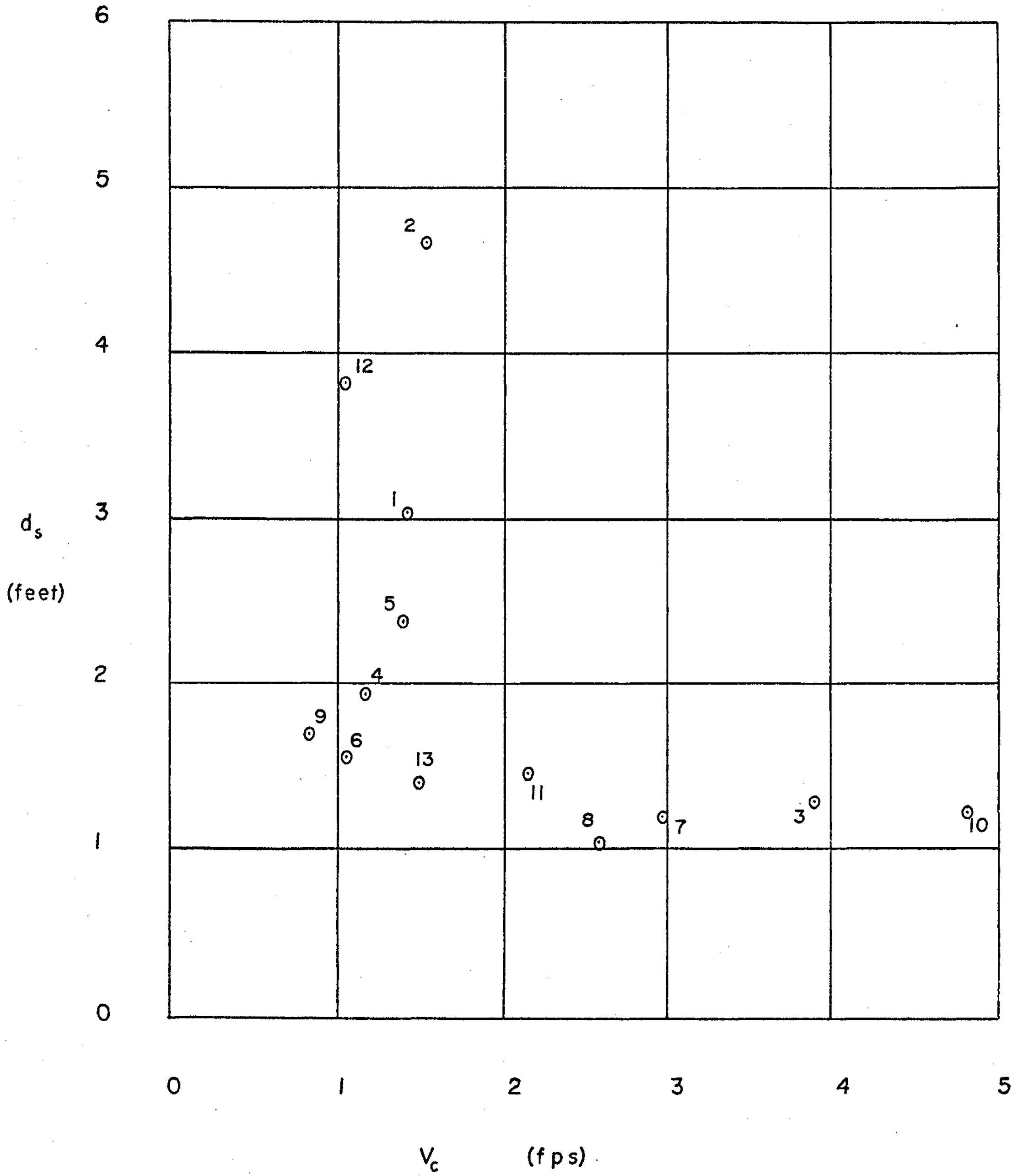


FIGURE 19 Relation of the depth of scour to the critical velocity at the bed of the rivers in Project A–I .

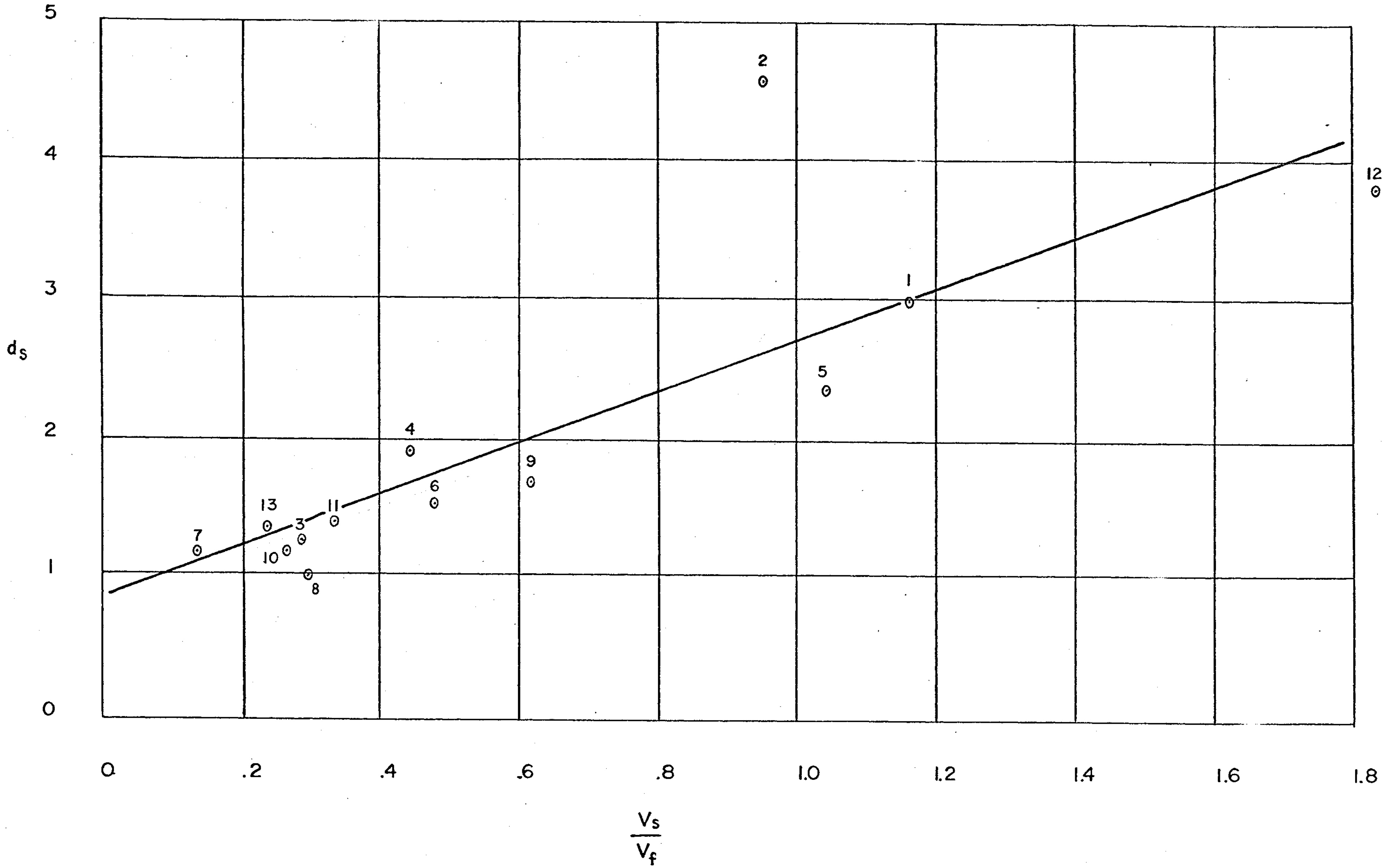


FIGURE 20 Graph to estimate the depth of scour. Plot of scour versus the V_s/V_f ratio.

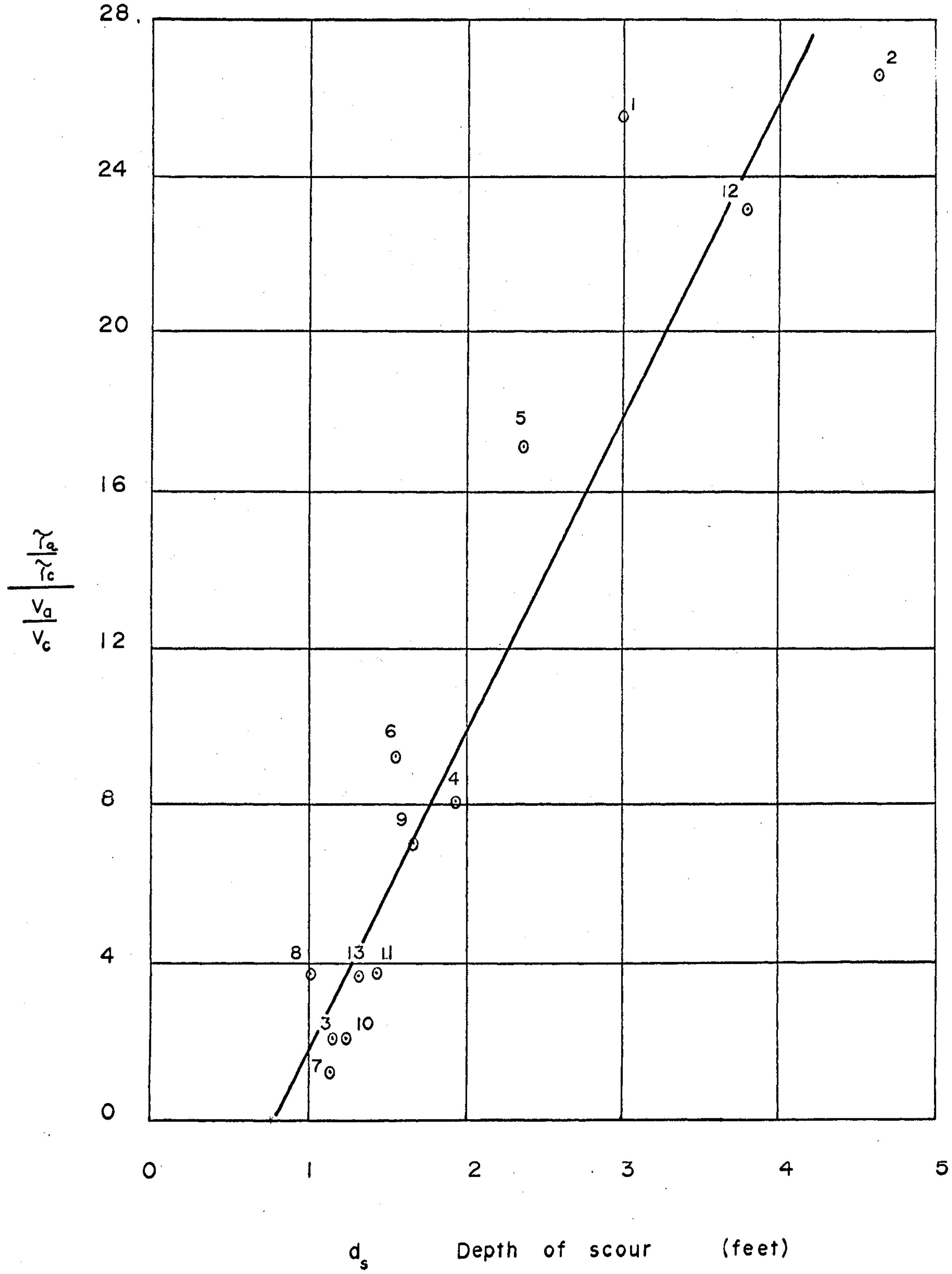


FIGURE 21 Graph to estimate the depth of scour. Plot of depth of scour versus the shear stress ratio over the velocity ratio.

REFERENCES

(1) Ahmid, Nazir, "Mechanism of Erosion Below Hydraulic Works." Proceedings of Minnesota International Hydraulics Convention, September 1-4, 1953.

(2) Albertson, M., Barton, J., and Simons, D. Fluid Mechanics for Engineers. Prentice-Hall Inc. pp. 454, 1960.

(3) Anderrson, S., "Stability of Armour Layer of Uniform Stones in Running Water". Proceedings of the Swedish Geotechnical Institute, Supplement No. 6, Stockholm, 1964.

(4) Blench, T. S., "Hydraulics of Canals and Rivers of Mobile Boundary". Civil Engineering Reference Book. Second Edition, Vol. 2, 1965.

(5) Chow, V. T. Open Channel Hydraulics. McGraw-Hill Book Co. Inc., pp. 164-179, 1959

(6) Hedar, P. A., "Filters as Protection Against Erosion". Proc. of the Swedish Geotechnical Institute, Supplement No. 6, Stockholm, 1964.

(7) Henner, R. G. and Ekse, M. I. Fundamentals of Transportation Engineering. McGraw-Hill Book Co. Inc., pp. 358-382. 1955.

(8) Karuks, E., "Prediction of Daily Average Flood Flows for Southern Ontario District Rivers". Ontario Joint Highway Research Program Report, No. 21, 1964.

(9) Keulegan, G. H., "Laws of Turbulent Flow Open Channels". Journal of Research of the National Bureau of Standards, Vol. 21, Research Paper, RP 1151, 1938.

(10) Lane, E. W. and Carlson, E. J., "Some Factors Affecting the Stability of Canals Constructed in Coarse Granular Materials." Proceedings of Minnesota International Hydraulics Convention, September 1-4, 1953.

(11) Laursen, E. M. and Toch, A., "Scour Around Bridge Piers and Abutments". Iowa Highway Research Board, Bulletin No. 4, 1956.

(12) Laursen, E. M., "Scour at Bridge Crossings". Iowa Highway Research Board, Bulletin, No. 8, 1958.

(13) Linsley, R. K., Kohler, M. A. and Paulhus, J. L. Applied Hydrology. McGraw-Hill Bk. Co. Inc., pp. 203-215, 1949.

(14) Neill, C. R., "River-Bed Scour, A Review for Bridge Engineers." Technical Publication, No. 23, Canadian Good Roads Assoc., 1964.

(15) Posey, C. J., "Some Basic Requirements for Protection Against Erosion." Proc. of Minnesota International Hydraulics Convention, September 1-4, 1953.

(16) Reinius, Erling, "Model Studies of the Erosion at a Bridge Site." Institution of Hydraulics at Chalmers University, Bulletin No. 7, 1956.

(17) Sanden, E. J., "Scour at Bridge Piers and Erosion of River Banks." Department of Highways, Province of Alberta, 1960.

(18) Stebbings, John, "The Shapes of Self-Formed Model Alluvial Channels." I.C.E. Proc., Paper No. 6642, 1963.

(19) Straub, L. G., "Dredge Fill Closure of Missouri River at Fort Randall." Proc. of Minnesota International Hydraulics Convention, September 1-4, 1953.

(20) Tison, L. J., "Studies of the Critical Tractive Force for the Entrainment of Bed Materials." Proc. of Minnesota International Hydraulics Convention, September 1-4, 1953.

NOMENCLATURE

a	effective area of a particle.
A	cross section area of the river.
b	river width at about half-depth.
B	river width.
C	bed-load charge in parts per hundred thousand (by weight).
D_{10}	maximum particle size, taken as the 10% retained value (by weight).
D_{50} D_{35} D_{25}	particle size respectively at the 50, 35 and 25 percent size retained by weight value.
d	depth of water measured from the surface to the river bed.
d_s	depth of scour at a river bed.
F_b	bed factor.
g	acceleration due to gravity.
n	Boundary roughness coefficient.
N_F	Froude Number.
N_R	Reynolds Number.
q	unit discharge.
Q	Discharge
R_H	Hydraulic Radius
S	Channel Slope
V_a	actual velocity
V_c	critical velocity
V_s	shear velocity based on shear stress
V_f	fall velocity based on particle size

ϕ	angle of repose of soil particles .
γ_s	specific weight of the soil.
γ_w	specific weight of the fluid.
μ	dynamic viscosity.
ν	Kinematic viscosity of fluid.
ρ	Density.
τ_a	actual boundary shear stress.
τ_c	critical boundary shear stress.

VITA AUCTORIS

1939 Lawrence Michael Brennan was born in Windsor, Ontario, Canada, on October 25, 1939.

1945 In September, 1945, he entered elementary school.

1953 In September, 1953, he enrolled at Assumption High School, Windsor, where he obtained his secondary education.

1957 In September, 1957, he enrolled in Pre-Engineering at Assumption University of Windsor.

1962 In October, 1962, he was graduated from Essex College with a Bachelor of Applied Science Degree in Civil Engineering.

1963 In September, 1963, he enrolled at the University of Windsor, in order to obtain the degree of Master of Applied Science in Civil Engineering.